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LEADING CONTENTS

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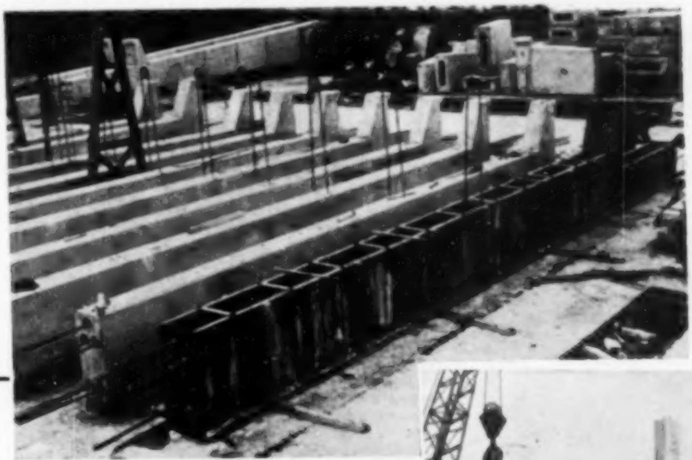

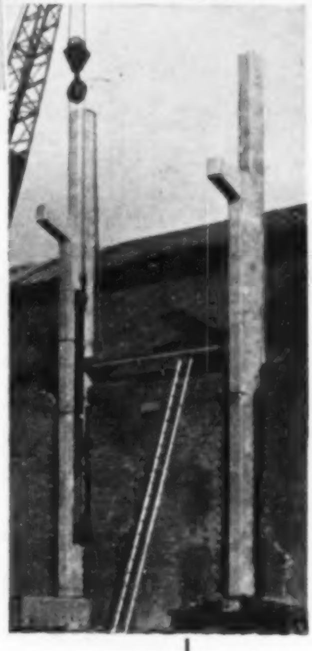

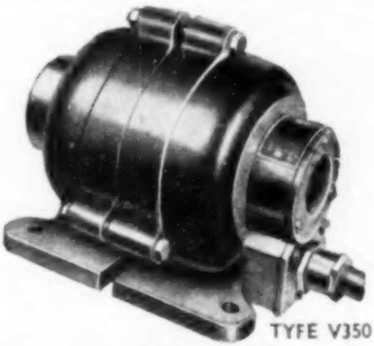


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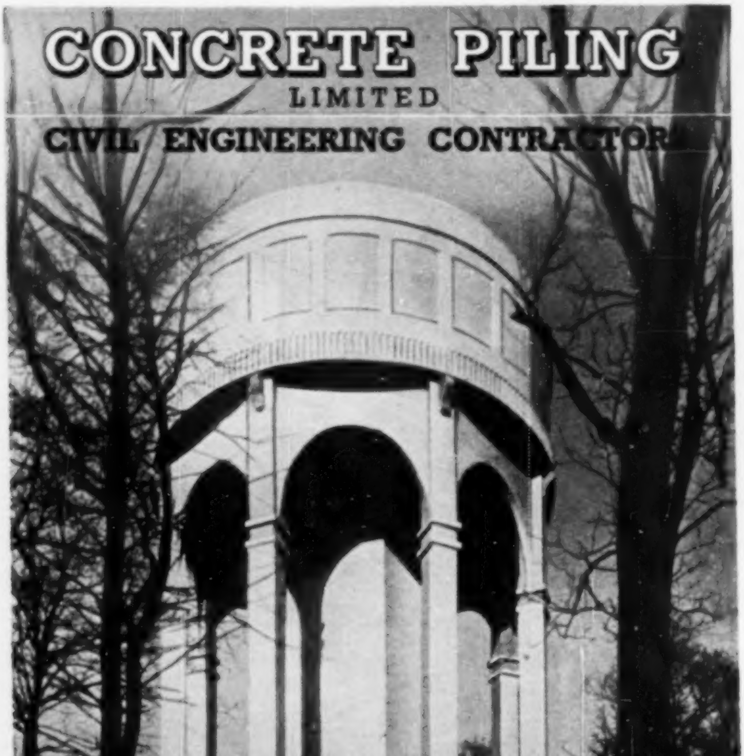
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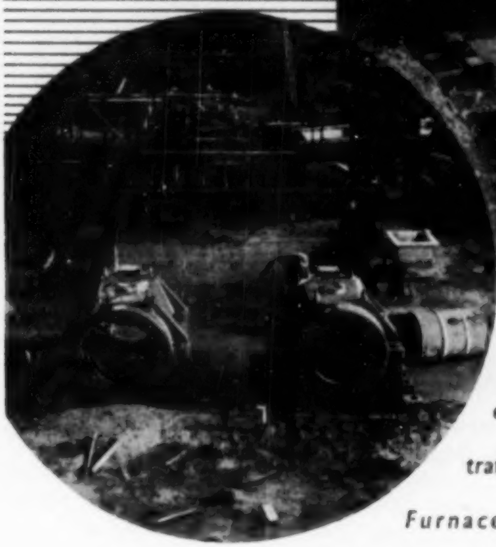

Reconstruction of Round Oak Steel Works, Brierley Hill. Chief Engineer : H. Shaw-Jones, Esq.

TARSLAG have been entrusted with all the Civil Engineering Work on this £4,000,000 Project. Illustration shows foundations for Melting Furnace Plant. Contract includes 250,000 cu. yds. of excavation, 15,000 cu. yds. of concrete, etc.

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CONCRETE
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Volume XLV. No. 9. LONDON, SEPTEMBER, 1950

EDITORIAL NOTES

Notions of Safety.

SOME papers have recently been published on the Continent suggesting that it might be desirable to consider the safety of structures from the point of view of the probabilities of the design load being attained or exceeded, of errors in the calculations, of dimensional errors in construction, and of weakness of the materials.

Investigations have been made of the probability of weakness in materials by statistical study of the dispersal curves of test results. A French engineer has estimated that the probability of a cube of ordinary concrete not resisting a pressure of about 800 lb. per square inch is between 1 to 100 and 1 to 1000. In the final report of the last congress of the International Association of Bridge and Structural Engineering an example is given of a concrete the probability of the strength of which being less than 1000 lb. per square inch is one in many millions; that it will not attain a strength of 2000 lb., one in 100,000; 3000 lb., one in 100; 4000 lb., one in five; and 4400 lb. per square inch, one in two. It is therefore suggested that a strength of not more than 4000 lb. per square inch might be reasonably assumed for the purpose of design.

It is improbable that a structure having the same factor of safety throughout can be designed or built. Therefore there are parts in which the stresses exceed or are less than the average or design stress, but the strength of a monolithic structure, in which local stresses may exceed considerably the average stress, is not the strength of its weakest member because inelastic behaviour of the overstressed material reduces the danger of failure. The rate of loading must not, however, exceed the rate of yielding, otherwise the relief upon which safety depends is not obtained. Likewise, the strength of an occasional weak patch of concrete may not necessarily determine the strength of a beam in which it occurs, as the excessive strain that may occur at the weak part may call into play the reserve of strength of a part that is stronger than need be. It is important that accidental weakness must not also occur at the latter part. The probability of the incidence of weakness, which is more likely to occur in a long than in a short member, must therefore be considered.

A simple example of the application of probabilities to the loads to which a structure is likely to be subjected is the case of a structure which is expected to have a useful life of, say, fifteen years, and which is on a site where it may be exposed to an exceptionally high wind which blows, on average, on one day in

twenty years. Also the structure may be subjected to an abnormal load once in its lifetime and must be designed to resist it. The probability of the high wind blowing during the useful life of the structure is 3 to 4, which amounts to almost a certainty. The structure should therefore also be designed to resist the high wind. The probability of the wind and the abnormal load occurring at the same time, is, however, one in many millions, which means that the occurrence is most unlikely, and it is therefore unreasonable to design the structure to resist both effects simultaneously. The calculated probability depends obviously upon the reliability of the data. The reliability of records of natural phenomena extending over many years is often dubious, and engineers are not likely to consider that the recorded severity or frequency is much of a guide to future happenings since there is much evidence to the contrary.

An important factor is the degree of probability that is critical; that is, at what odds is it advisable to take into account a chance effect? In the foregoing example there is no certainty that the abnormal load and the great gale will not occur at the same time, although it is not likely that this will happen. But the extremes of " almost a certainty " and " most unlikely " are not always the case. That dead loads and most hydrostatic loads will attain their assumed values is certain, as also may loads on warehouse floors, but the occurrence of standard live loads on bridges and the incidence of the assumed superimposed loads on the floors of some buildings is less likely. It must be noted, however, that the uniformly distributed loads assumed to be the superimposed loads on floors do not resemble the actual probable loads, but are loads that may have the same effect as the actual loads. A smaller factor of safety for live loads than that adopted for the dead load or for hydrostatic loads may be permissible.

The effect of a load operating occasionally for a short time on a reinforced concrete structure may be different from that of the same load acting permanently (as does the dead load) or for a long while, or if it acts at frequent intervals. The occurrence of an abnormal load on a bridge is not in quite the same category, because either the bridge is designed for the occasional passage of such a load, or vehicles imposing loads in excess of the design loads are prohibited from using the bridge. But the probability of the effect of an abnormal load being transient is reflected in the practice of permitting greater stresses when an abnormal load occurs. A similar increase is permitted when considering wind pressure, and the improbability of a wide bridge being fully loaded is often taken into account in the same way that a slab of a floor is often designed for a larger load than are the beams supporting the slab.

A complete consideration of probabilities as affecting the safety of a structure seems to raise many new problems, an effect not uncommon in advances in scientific study. It is difficult to assess what odds can be safely accepted. The data required to enable useful results to be obtained in structural engineering seem to be beyond immediate attainment and so, while notions of safety based on chance may attract the statisticians, most engineers will no doubt continue to think of the factor of safety as the ratio between the well-defined, but assumed, working load, and the less well-defined and almost incalculable load that causes failure of a structure.

The Design of Prestressed Concrete Beams from Fundamental Principles.

By J. W. H. KING, M.Sc., A.M.Inst.C.E., A.M.I.Struct.E.

THE calculation of the stresses in prestressed concrete beams is generally based on the well-known formula $\frac{M}{I} = \frac{f}{y}$, steel embedded in the concrete being considered to be replaced by an area of concrete equal to the modular ratio times the area of the steel. It seems that a major objection to this method, in addition to the complex formulæ and notation often necessary, is that the modular ratio is not definable since the concrete and steel are stressed beyond the rectilinear stress-strain range. It is also commonly implied that tensile strains in the concrete produce tensile stresses, which may not always be so. The effects of shrinkage, creep, and hysteresis are not readily perceptible, and structures embodying precast prestressed beams and cast-in-situ slabs of concrete of different qualities are difficult to analyse accurately.

The method of design described overcomes these objections and is based

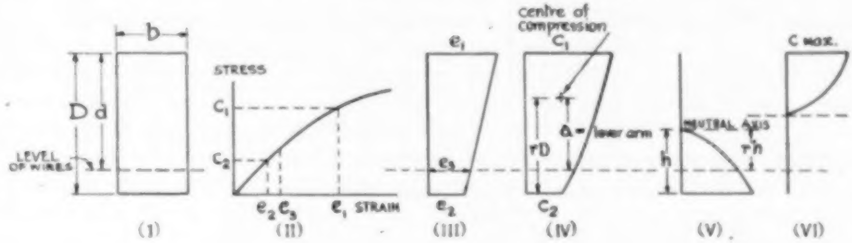


Fig. 1.

on only the principles and fundamental data which ought to be known or inferred before starting to design a beam. The data required include stress-strain curves of the concrete and the steel wires, thereby permitting the direct conversion of strain to stress without employing single values of the modular ratio or elastic moduli of the concrete and wires. (Although steel wires are referred to throughout this article, the method applies equally to steel bars.) The basic principles are those of simple bending, that is, plane sections remain plane, the total internal compressive force equals the total internal tensile force at any cross section, and the value of the couple produced by these forces equals the applied bending moment. Advantages of the method are that notation and formulæ are unnecessary, and what takes place at the section at all stages, and the effects of change in size or shape, sustained loads, and variation of the stress-strain relationship of the materials, can readily be seen. It is possible, therefore, to use the materials to the best advantage.

Three types of prestressed concrete beam are generally considered, namely, (1) Beams in which the wires are stretched before the concrete is placed and which are bonded to the concrete; (2) Beams in which the wires are stretched after the concrete has hardened, the wires being anchored at the ends of the

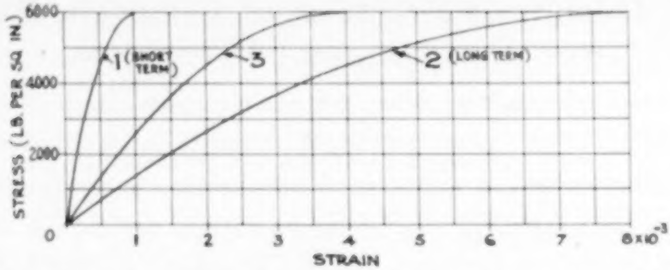


Fig. 2.—Stress-strain Curves for Concrete having a Strength of 6000 lb. per square inch.

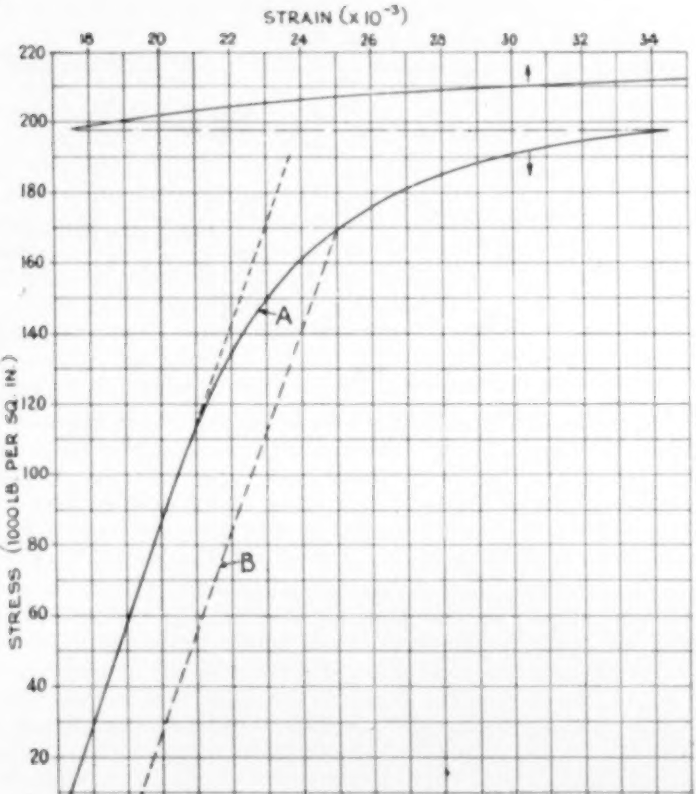


Fig. 3.—Stress-strain Curve for Hard Drawn Steel Wire.

beam and not bonded to the concrete; (3) Beams as in (2) but in which the wires are bonded to the concrete, generally by grouting. The principle of the proposed method is illustrated by the rectangular beam at (i) in *Fig.* 1. The stress-strain curve is assumed to be as at (ii). A bending moment due to a load may produce a rectilinear distribution of strain as at (iii), and reference to (ii) gives the distribution of stress as at (iv). The strain e_3 of the concrete at the level of the wires can be determined by simple proportion from the strains e_1 and e_2. The total compression C on the section, which is b times the area of the stress diagram at (iv), must be balanced by the total tension T in the wires, whence a suitable area of and stress in the wires can be ascertained. T or C multiplied by a must equal the applied bending moment. At the time precompression is induced in the beam, that is at the stage called " transfer ", the distribution of stress might be as at (v) if the concrete is assumed not to resist tension. If the effect of the weight of the beam is neglected, at transfer no external bending moments are applied to the beam; therefore the centroid of the stress diagram coincides with the centre of the wires. The total compressive and tensile forces are, of course, equal. For conditions at the transfer stage and under external load to be compatible, the change of strain of the wires and concrete from one stage to another must be the same, either locally if the wires are bonded or over the entire length of the beam if the wires are not bonded. These are the only matters to be considered, and provided that they are complied with at two critical stages, say, at transfer and at failure, the conditions at intermediate stages are of no importance in themselves, although of possible value in modifying the stress-strain curves to allow for creep or hysteresis.

A beam can be designed conveniently by considering first the conditions at failure, which occurs at a bending moment that is some multiple of the bending moment due to the working load. At failure the conditions of stress may be as at (vi) in *Fig.* 1, the maximum concrete stress $c_{max.}$ being the compressive strength of the concrete. The stress in the wires in a rectangular beam at failure can generally be assumed to be about the 0·2 per cent. proof-stress, since beyond this stress the strain of the wires increases so much for a small increase in stress that, as failure is approached, the neutral axis rises and the condition is reached where the maximum compressive stress in the concrete equals the compressive strength.

For a beam to be as small as possible it is necessary to make the best use of the concrete and, therefore, at failure the neutral axis should be as low as possible. This condition, however, generally implies a large amount of steel and a high stress in the concrete at transfer. In a good design this must be taken into account. It should be noted that it is possible to have almost any desired condition in the concrete at whatever stage is first considered.

Examples.

The foregoing points will be made clear in the examples which follow. Unless otherwise stated the beam is type (3), the bending moment due to the weight of the beam and the tensile resistance of the concrete are ignored, the cavity containing the wires (or the displacement of the concrete by the wires) is neglected, and the cross section of the beam is constant. Therefore at transfer

the strain of the concrete at the level of the wires and the strain of the wires are constant throughout the length of the beam.

The approximate method used to calculate the area and position of the centroid of the stress diagrams is as follows: If the ordinates of a continuous curve are α, β, and γ at $x = 0$, $x = \frac{D}{2}$, and $x = D$ respectively, the area under the curve is $\frac{D}{6}(\alpha + 4\beta + \gamma)$, and the centroid is at $x = D\left(\frac{2\beta + \gamma}{\alpha + 4\beta + \gamma}\right) = rD$ in *Fig.* 1 (iv). The moment of the area about the origin is $\frac{D^2}{6}(2\beta + \gamma)$.

The stress-strain curves assumed for the concrete and wires are given in *Figs.* 2 and 3 respectively, the latter being typical of the hard drawn wire of 0·2 in. diameter generally used. The curves give general extensions and not

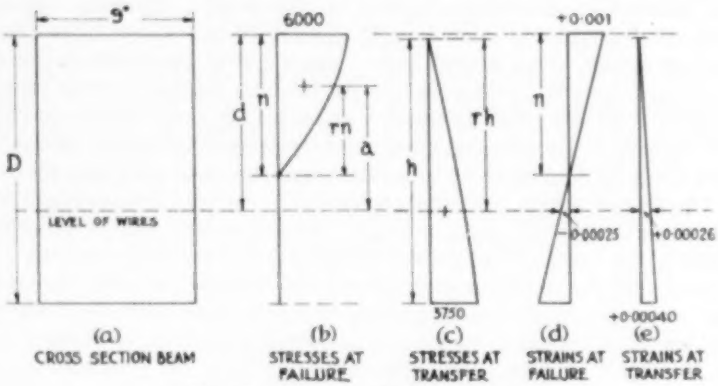


Fig. 4.—Example Nos. 1 and 2.

the larger local extensions near the point of failure. The stress-strain curves for concrete are typical of concrete having a crushing strength of 6000 lb. per square inch and are based on tests at 28 days. The fact that creep occurs immediately a load is applied makes it almost impossible to obtain the "short-term" curve (1) by ordinary methods. The use of this curve, which has been derived from (3), generally leads to extra safety because the concrete is considered to be more rigid. Since strains tend to be smaller if the concrete is stronger or older than specified, curve (1) denotes a stress-strain relationship which is as high as may generally be encountered. The "long-term" curve (2) is reasonable for the concrete described. The intermediate curve (3) is typical for loads sustained for an hour or two. The "short-term" curve is applicable to very rapid loading, and the "long-term" curve includes the full effect of creep.

EXAMPLE NO. 1. RECTANGULAR BEAM.—A beam 9 in. wide (*Fig.* 4*a*) is to resist a bending moment of 806,400 in.-lb. at the working load. The factor of safety is to be 2·5. The permissible stress in the concrete at transfer is 3750 lb. per square inch, and 160,000 lb. per square inch in the wires at failure of the beam.

At failure. To obtain a light member it will be assumed that at failure the position of the neutral axis is such that $n = 0{\cdot}8d$, and the distribution of stress in the concrete is therefore as in *Fig. 4b*. From curve No. 1 (*Fig. 2*), if the maximum stress in the concrete is 6000 lb. per square inch, the total compression C is

$$\frac{9n}{6}\,[0 + (4 \times 4500) + 6000] = 36{,}000n = 28{,}800d.$$

$$\text{Lever arm} = 0{\cdot}2d + 0{\cdot}8d\left[\frac{6000 + (2 \times 4500)}{6000 + (4 \times 4500)}\right] = 0{\cdot}7d.$$

The total moment of resistance must be 2·5 times the working bending moment, that is,

$$28{,}800 \times 0{\cdot}7d^2 = 2{\cdot}5 \times 806{,}400\,; \quad \text{therefore } d = 10 \text{ in.}$$

The strain of the concrete at the top of the beam is 0·001, and at the level of the wires is $-\frac{2}{8} \times 0{\cdot}001 = -\,0{\cdot}00025$ (*Fig. 4d*).

At transfer. The distribution of stress in the concrete is as in *Fig. 4c*. Therefore

$$r = \frac{(2 \times 2100) + 3750}{0 + (4 \times 2100) + 3750} = 0{\cdot}65.$$

$$C = \frac{9h}{6}[0 + (4 \times 2100) + 3750] = 18{,}225h.$$

The strain of the concrete at the bottom of the beam, from curve No. 1 (*Fig. 2*), is 0·0004, and therefore at the level of the wires it is 0·0004 × 0·65 = 0·00026 (*Fig. 4e*). The change of strain of the wires between transfer and failure is therefore 0·00026 − (− 0·00025) = 0·00051. But at failure the strain of the wires corresponding to 160,000 lb. per square inch is 0·00685 (*Fig. 3*). Hence the strain after transfer is 0·00685 − 0·00051 = 0·00634, which corresponds to a stress in the wires of 154,500 lb. per square inch, which is the tensile stress which must be induced in the wires when they are stretched. The cross-sectional area of the wires required is given by

$$\frac{C}{160{,}000} = \frac{28{,}800 \times 10}{160{,}000} = 1{\cdot}8 \text{ sq. in.}$$

The total compression at transfer is therefore

$1{\cdot}8 \times 154{,}500 = 18{,}225h$ lb.; therefore $h = 15{\cdot}25$ in., and $rh = 0{\cdot}65h = 9{\cdot}9$ in. Therefore the wires should be 15·25 − 9·9 = 5·35 in. from the bottom of the beam. The total depth D of the beam is therefore 10 + 5·35 = 15·35 in. It would be difficult to place 1·8 sq. in. of 0·2-in. wires in this beam, and to do so it is necessary to widen or deepen the beam considerably at the bottom. The alternative of a greater stress in the wires, say, 200,000 lb. per square inch at failure, reduces the area of the wires to 1·44 sq. in., the strain of which at failure would be 0·019 and at transfer 0·01849; at the latter strain the stress is still about 200,000 lb. per square inch. Hence the compressive force on the concrete at transfer is increased, and the risk of failure due to creep in the wires and concrete, or to errors in stretching, is increased. A stress of 3750 lb. per square

inch in the concrete at transfer is high. A better solution is to assume a smaller value of n at failure as in Example No. 2.

EXAMPLE NO. 2. ALTERNATIVE DESIGN OF RECTANGULAR BEAM.—Assume that at failure $n = 0{\cdot}4d$ and that the stress in the concrete at transfer must not exceed 2200 lb. per square inch, a stress more commonly used than 3750 lb. per square inch as in Example No. 1.

At failure.

Total compression $= 36{,}000n = 14{,}400d$. Lever arm $= 0{\cdot}6d + \left(\frac{5}{8} \times 0{\cdot}4d\right)$ $= 0{\cdot}85d$.

Equating moments, $14{,}400 \times 0{\cdot}85d^2 = 2{\cdot}5 \times 806{,}400$; therefore $d = 12{\cdot}8$ in.

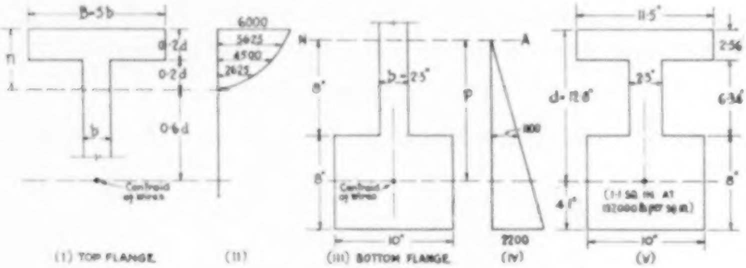


Fig. 5.—Example No. 3.

The strain in the concrete at the level of the wires is $-0{\cdot}001 \times \frac{6}{4} = 0{\cdot}0015$.

The area of wires required is $\frac{14{,}400 \times 12{\cdot}8}{160{,}000} = 1{\cdot}152$ sq. in.

At transfer.

$$C = \frac{9h}{6}[0 + (4 \times 1125) + 2200] = 10{,}050h\ ; \quad r = \frac{2200 + 2250}{2200 + 4500} = 0{\cdot}662.$$

The maximum strain of the concrete is $0{\cdot}00021$, and the strain of the concrete at the level of the wires is $0{\cdot}00021 \times 0{\cdot}662 = 0{\cdot}00014$. The change of strain of the wires is $0{\cdot}00014 + 0{\cdot}0015 = 0{\cdot}00164$, and the strain of the wires at transfer is $0{\cdot}00685 - 0{\cdot}00164 = 0{\cdot}00521$, corresponding to a stress of 138,000 lb. per square inch, which is the stress to which the wires should be stretched. Hence $10{,}050h = 1{\cdot}152 \times 138{,}000$; therefore $h = 15{\cdot}8$ in. ; $rh = 0{\cdot}662 \times h = 10{\cdot}45$ in. ; and $h - rh = 5{\cdot}35$ in.

The wires should therefore be 5·35 in. from the bottom of the beam. The depth of the beam is $12{\cdot}8 + 5{\cdot}35 = 18{\cdot}15$ in., which gives a larger beam than that in Example No. 1. Alternatively an I-beam, as in Example No. 3, could be used.

EXAMPLE NO. 3. I-BEAM.—An I-beam will be designed and compared with the rectangular beam in Example No. 2. It is assumed that the proportions of the top flange are as in *Fig.* 5 (i), the stresses and the depth to the neutral axis at failure being as nearly as possible the same as in Example No. 2. The size of the bottom flange will be determined by the calculation.

At failure, the distribution of stress in the top part of the beam will be as *Fig.* 5 (ii). The total compression is

$$\frac{0\cdot 2bd}{6}[6000 + (4 \times 5625) + 4500]5 + \frac{0\cdot 2bd}{6}[4500 + (4 \times 2625) + 0]$$
$$= 5500bd + 500bd = 6000bd.$$

The area of wires required is $\frac{6000bd}{160{,}000}$ sq. in. The moment, about the wires, of the compressive stresses in the flange is

$5500bd$ (distance of centroid of compressive area from the wires)
= (the moment about the bottom edge of the top flange) + ($5500bd \times 0\cdot 8d$),

$$= \frac{(0\cdot 2d)^2}{6}[6000 + (2 \times 5625)]5b + 4400bd^2 = 4975bd^2.$$

Similarly the moment about the wires of the compression in the web is

$$\frac{(0\cdot 2d)^2}{6}[4500 + (2 \times 2625)]b + (500 \times 0\cdot 6bd^2) = 365bd^2.$$

Hence the total moment is $5340bd^2$, which must equal $2\cdot 5 \times 806{,}400$, whence bd^2 is 377. If d is assumed to be the same as in Example No. 2, that is 12·8 in., b is 2·3 in. and the width of the top flange is 11·5 in. The area of wires required is 1·1 sq. in., which is about the same as in Example No. 2.

Conditions at transfer determine the size of the bottom flange. A suitable trial size can be determined by assuming the flange to be stressed uniformly at the maximum stress of 2200 lb. per square inch, and to resist a compression equal to the total tensile force of 1·1 sq. in. × 160,000 lb. per sq. in. in the wires. The area required is 80 sq. in., say, 10 in. by 8 in. as in *Fig.* 5 (iii). Assume also that at this stage the neutral axis is 8 in. above the bottom flange. Since the maximum stress in the concrete is only 2200 lb. per square inch the distribution of stress may be assumed to be rectilinear, as in *Fig.* 5 (iv), from which the total compression is

$$\left(10 \times 16 \times \frac{2200}{2}\right) - \left(8 \times 7\cdot 7 \times \frac{1100}{2}\right) = 176{,}000 - 33{,}880 = 142{,}120 \text{ lb.}$$

The distance p from the neutral axis to the centre of compression (which must coincide with the position of the wires) is $\dfrac{\left(176{,}000 \times \frac{16 \times 2}{3}\right) - \left(33{,}880 \times \frac{8 \times 2}{3}\right)}{142{,}120}$
= 11·9 in. The neutral axis is therefore 0·9 in. below the top of the beam and lies in the top flange. The small compressive resistance of that part of the top flange below the neutral axis and outside the web has been ignored. The overall depth of the beam is 16 + 0·9 = 16·9 in., and the complete cross section is as in *Fig.* 5 (v), which results in a beam about 25 per cent. lighter than that in Example No. 2.

To complete the design it is necessary to check the stress in the wires at transfer and to ensure that the changes of strain in the wires and concrete are the same. The strain of the concrete at the level of the wires is $\frac{11\cdot 9}{16}$ times the

maximum strain of 0·00021, that is 0·00016. Therefore the strain of the steel at transfer (from Example No. 2) is 0·00685 − 0·00150 − 0·00016 = 0·00519, which corresponds to a stress of 137,000 lb. per square inch and a total force of 140,700 lb., which is not exactly equal to the compression of 142,120 lb. A maximum stress of 2180 lb. per square inch in the concrete results in a compression of 140,700 lb. and does not greatly affect the correlation of strain.

The Effects of Various Factors.

CREEP.—The effect of creep, in cases where at failure the neutral axis is above the wires, is to increase the stress in the wires, and to a less extent the moment of resistance, at failure. It is therefore safer to use the "short-term" curve for the concrete (*Fig.* 2) for conditions at failure. This curve should be used also for conditions at transfer if the process of transfer is rapid, but if the process takes several hours the use of the intermediate curve No. 3 may be justified. The effect is, however, small if the strain of the concrete at transfer is small. The depth of the beam in Example No. 2 would be reduced to 17·9 in. (compared with 18·15 in.), and the initial stress required in the wires to 130,000 lb. instead of 138,000 lb. per square inch by using curve (3) at transfer. Reductions of a similar degree would apply to Example No. 1.

STRESS-STRAIN RATIO OF CONCRETE.—There is also some advantage if the "short-term" curve for the concrete used shows, for equal stresses, strains greater than those indicated by curve No. 1. The effect of "short-term" strains of, say, twice those of curve No. 1 can be shown by recalculating Example No. 2, and is to reduce the stretching force to 117,000 lb. and the depth of beam to 16¾ in. Advantage cannot be taken of such reductions unless the concrete to be used is definitely known to be at least as deformable as is indicated by the curve used. The advantage of having accurate data in this respect is apparent.

ERRORS IN TENSIONING.—The effect of the tension in the wires being other than that required by the calculations can be seen approximately by recalculating Example No. 1 assuming that the wires are stretched so that the maximum compressive stress in the concrete is only 2200 lb. per square inch, as in Example No. 2, instead of 3750 lb. per square inch as intended. At transfer, $r = 0{\cdot}662$ in Example No. 2; therefore for the beam in Example No. 1,

$h = \dfrac{5{\cdot}35}{0{\cdot}338} = 15{\cdot}75$ in., $C = 10{,}050h = 159{,}000$ lb.; the stress in wires having a cross-sectional area of 1·8 sq. in. is 88,300 lb. per square inch and the strain 0·00305, instead of 154,500 lb. per square inch and 0·00634 as intended. The strain of the concrete at the level of the wires is 0·00014 as in Example No. 2.

Conditions at failure must generally be determined by trial. Assuming a position for the neutral axis, the total compression is $36{,}000n$, as in Examples Nos. 1 and 2 at failure, and the strain of the concrete 0·001 at the top of the beam. The change of strain between transfer and failure at the level of the wires is $0{\cdot}00014 + 0{\cdot}001\left(\dfrac{10 - n}{n}\right)$. Reference to *Fig.* 3 shows that, for a small stress in the wires, a change of strain of 0·00035 corresponds to a change of stress of about 10,000 lb. per square inch, that is a force of 18,000 lb. in the

wires. Hence, equating tension and compression at failure, and assuming that the stress-strain relation of the wires is constant,

$$36{,}000n = \frac{18{,}000}{0{\cdot}00035}\left(0{\cdot}00014 + \frac{0{\cdot}01}{n} - 0{\cdot}001\right) + 159{,}000\text{; therefore } n = 5{\cdot}7 \text{ in.}$$

$$C = 36{,}000 \times 5{\cdot}7 = 205{,}200 \text{ lb.}$$

The strain of the steel is $0{\cdot}00305 + 0{\cdot}00014 + 0{\cdot}001\left(\frac{4{\cdot}3}{5{\cdot}7}\right) = 0{\cdot}00394$, which is just outside the rectilinear range and corresponds to the low stress of 112,000 lb. per square inch, the force in the wires being therefore 201,600 lb. A closer calculation gives $n = 5{\cdot}6$ in., the force in the wires 202,200 lb., and the lever arm $10 - \left(\frac{3}{8} \times 5{\cdot}6\right) = 7{\cdot}9$ in. The moment of resistance at failure is 1,595,000 in.-lb., compared with the required resistance of 2,016,000 in.-lb. In this example, therefore, a reduction of nearly 45 per cent. in the stretching force has affected the moment of resistance at failure by about 25 per cent.; consequently the small errors in tensioning that may occur are not likely to reduce seriously the factor of safety. Errors in tensioning may be due to inaccurately measuring the stretching force, as in the case considered, or because of broken wires. In the latter case, the small reduction in the total area of the wires due to one of several wires breaking would be compensated for partly by the greater strain of the wires consequent on the raising of the neutral axis. The increase of stress would not be as great as the increase of strain because of the reduced slope of the stress-strain curve. The percentage reduction in the moment would, however, generally be less than the percentage reduction in the area of the wires.

Allowance for Space Occupied by Wires.—No allowance is made in the foregoing calculations for the cavity in the concrete for the wires or for the concrete displaced by the wires. Since this omission only affects conditions at transfer, at which stage the local concrete stress is known, allowance can be made as follows.

In Example No. 2 the wires might require a cavity of about 3 sq. in. At transfer the stress in the concrete at the level of the wires is 1450 lb. per square inch (from curve No. 1, *Fig.* 2, for a strain of 0·00014). Therefore a compression of $3 \times 1450 = 4350$ lb. is missing from the total of $1{\cdot}152 \times 135{,}000 = 159{,}000$ lb. required. Hence $10{,}050h$ must be equal to $159{,}000 + 4350$, whence $h = 16{\cdot}25$ in. Since the centroid of the compressive area still coincides with the position of the wires (and the cavity), r is still 0·662, whence the depth of beam below the wires is 5·5 in., and the beam must be 18·3 in. deep to allow for a cavity of 3 sq. in.

Non-Rectangular Beam.—If the cross section of a non-rectangular beam cannot be divided conveniently into rectangles as in Example No. 3, a curve of (breadth × stress) should be drawn and the method of calculation of a rectangular beam of unit breadth applied. If the curve is irregular, more precise methods may have to be employed to determine the area and position of the centroid.

No Tensile Stress in the Concrete.

The design in Example No. 2 implies that tensile strain will be induced in the concrete at the top of the beam at transfer, and therefore cracking may

occur since the neutral axis is 2·35 in. below the top at this stage. If tensile strain is not permissible the depth of the beam should be increased to about 19·2 in., the maximum stress in the concrete at transfer then being about 1850 lb. per square inch. Alternatively stretched wires could be provided in the top of the beam.

The bending moment which would just cause tensile stress in the bottom of the beam can be determined as follows. First calculate the strain of the wires when there is no strain of the adjacent concrete. In Example No. 2 this strain is 0·00685 − 0·00150 = 0·00535. The strain when tension first develops at the bottom of the beam will be between 0·00535 and 0·00521, corresponding to a stress in the steel of about 138,500 lb. per square inch; the corresponding maximum stress in the concrete, assuming rectilinear distribution, is about

STRESS DIAGRAM Nº	1	2	3	4	5	6	7	UNITS
MIN. STRESS	0	0	0	200	500	700	1000	lb. per sq. in.
MAX. "	3700	2600	2000	1800	1500	1300	1000	"
HEIGHT OF CENTRE OF COMPRESSION	2·07	3·03	4·00	4·40	5·00	5·40	6·00	inches
LEVER ARM	0·07	1·03	2·00	2·40	3·00	3·40	4·00	inches
MOMENT OF RESISTANCE	5,040	74,200	144,000	172,500	216,000	245,000	288,000	in.-lb.
STRAIN OF CONCRETE AT LEVEL OF WIRES	2·06	1·56	1·25	1·12	1·00	0·90	0·75	$\times 10^{-3}$
FOR DIAGRAMS INVERTED:								
LEVER ARM	7·93	6·97	6·00	5·60	5·00	4·60	4·00	inches
MOMENT OF RESISTANCE	570,000	500,000	432,000	403,000	360,000	330,000	288,000	in.-lb.
STRAIN OF CONCRETE AT LEVEL OF WIRES	−2·06	−0·22	0·25	0·38	0·50	0·60	0·75	$\times 10^{-3}$

Fig. 6.

$\frac{138{,}500 \times 1{\cdot}152}{18{\cdot}15(9 \times 0{\cdot}5)}$ = 1950 lb. per square inch, which can be used for the first trial calculation, since *Fig. 2* shows that up to this stress rectilinear distribution is justified. Therefore r is 0·666, and the maximum strain of the concrete is 0·00018. The strain at the level of the wires is $\frac{5{\cdot}35}{18{\cdot}15} \times 0{\cdot}00018 = 0{\cdot}00005$, and the strain of the wires is 0·00535 − 0·00005 = 0·0053, corresponding to a stress of 138,500 lb. per square inch, which is the stress assumed; therefore the total compression equals the total tension. The lever arm is (0·666 × 18·15) − 5·35 = 6·75 in. The moment developed is 138,500 × 1·152 × 6·75 = 1,080,000 in.-lb., which is equal to the bending moment at which cracks occur and is about 25 per cent. more than the bending moment of 806,400 in.-lb. at the working load.

The foregoing is based on the "short-term" stress-strain curve (*Fig. 2*) for

concrete, but as a small overload may persist for a long time it is useful to examine the case using the "long-term" curve No. 2. With the trial values already calculated the maximum strain of the concrete is 0·00144; at the level of the wires the strain is 0·00042, the strain of the wires being 0·00535 − 0·00042 = 0·00493, corresponding to a stress of 133,500 lb. per square inch.

$$C = 18{\cdot}15 \times \frac{9}{2} \times 1950 = 159{,}000 \text{ lb.}; \quad T = 1{\cdot}152 \times 133{,}500 = 154{,}000 \text{ lb.}$$

Since C and T are not equal, a second trial calculation is made with a maximum stress in the concrete of 1900 lb. per square inch for which both C and T are 155,000 lb. Since the lever arm is still 6·75 in., the bending moment just producing tensile stress is 1,045,000 in.-lb., which is less than that calculated

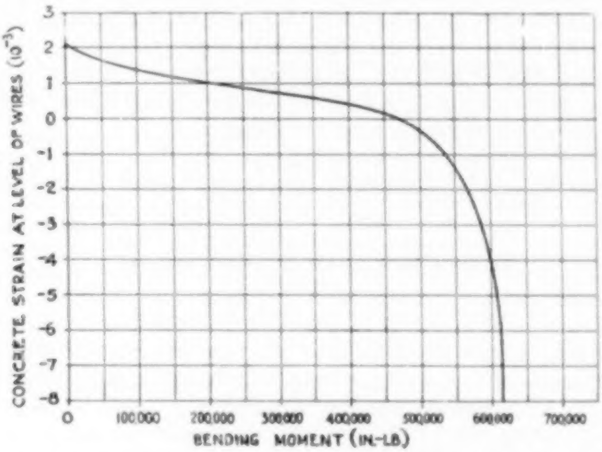


Fig. 7.—Variations of Strain with Bending Moment at Level of Wires.

from the "short-term" curve because of the reduction of the tension in the wires due to creep of the compressed concrete adjacent to them. The difference is small in this case, since the stress in the concrete at the level of the wires is small.

Wires Stretched before Embedment in Concrete.

The foregoing examples relate to beams in which the wires are stretched after the concrete has hardened, but if the wires are stretched before the concrete is placed around them the effect of the shrinkage of the concrete must be considered. Immediately before transfer, the concrete around the wires may be subjected to a tensile strain of, say, 0·0003 due to shrinkage, and hence the strain of the wires when stretched must be this much in excess of the strain required after transfer. The change of strain between transfer and failure may also be decreased by shrinkage by, say, 0·0002. Proceeding as before from conditions of failure to the requirements at stretching, the alterations of the calculated effects in Example No. 2 are that the change of strain of the wires between transfer and failure is 0·00164 − 0·0002 = 0·00144; the strain of the wires after transfer is 0·00541, the corresponding stress being 142,000 lb. per square inch. The value of h required is 16·3 in. and the total depth of the beam

should be 18·3 in. instead of 18·15 in. To allow for the shrinkage before transfer, the initial tensile strain in the wires must be 0·00541 + 0·0003 = 0·00571, necessitating a stress of 146,000 lb. per square inch instead of 138,000 lb. per square inch. The effect of shrinkage increases as the neutral axis at failure is lowered, and as the elastic modulus of the concrete increases.

HYSTERESIS IN STEEL WIRES.—If the wires are stretched before the concrete is cast around them, the tensile stress in the wires before transfer may be greater than the stress at failure, particularly if the wires are initially overstressed to allow for creep of the steel. In such cases the strain of the wires may be partly elastic and partly plastic, subsequent reduction of strain leaving the plastic strain unaltered. Curve B in *Fig.* 3 is parallel to the initial rectilinear curve A and shows the hysteresis effect. Curve B applies to steel which has been stressed to 170,000 lb. per square inch and should be used for subsequent lower stresses in cases where the wires have been previously stressed to this amount. Curves similar to B can be drawn for other values of overstress, and if used instead of the curve A (up to the point of initial stress) the calculations given in the foregoing need no further alteration.

The stress at failure in wires which are stretched after the concrete has hardened is always greater than that at transfer, and hysteresis effects do not arise.

Beams with Non-bonded Wires.

The assumption that the strains of the wires and concrete at the level of the wires are at transfer constant throughout the length of a beam in which the wires are straight and are stretched after the concrete has hardened, is justified if the bending moment due to the weight of a beam of uniform cross section is not considerable. Subsequent bonding ensures that local changes of strain are the same in the wires and concrete, and the foregoing methods of calculation apply. In the case of non-bonded wires in a loaded beam the strain of the wire is constant throughout its length at each stage of loading, but the strain of the concrete at the level of the wires varies from point to point, and compatibility must be based on total extensions or contractions and not on local strains. The fundamental rules, that the total compression equals the total tension, and the total compression (or tension) multiplied by the lever arm equals the local bending moment, still apply. In addition, since the tensile force is everywhere the same, the area of the compression diagram must be the same at all sections of the beam; consequently the lever arm varies as the bending moment. In *Fig.* 6 are shown various distributions of compressive stress to give a force of 72,000 lb. in a 12-in. by 6-in. beam with non-bonded wires placed 2 in. from the bottom. The resulting lever arms, moments of resistance, and strains of the concrete at the level of the wires are also given. It is seen that strain and moment of resistance do not vary in proportion, so that the assumption that the mean strain in the concrete at the level of the wires is equal to the strain at the section of mean bending moment is generally untrue. One way to find the strain in the concrete at the level of the wires is to draw a curve of strain against moment of resistance as in *Fig.* 7, and obtain therefrom, and from the known distribution of bending moment along the beam, a curve showing strain in relation to position along the beam. From the curve the mean strain can be determined,

and this must correspond to the mean strain of the wires under the same condition of loading. The consequent calculations are laborious. The writer does not recommend beams with non-bonded wires because overloading is likely to produce wide cracks, and may in some cases produce sudden failure due to the lever arm being reduced by upward movement of the wires in the cavity.

Correspondence.

ENGINEERING EDUCATION.

SIR,—I feel bound to comment on the excellent Editorial Notes on Engineering Education and Training in your numbers for January and May. It is deplorable that some engineers are so narrow in their outlook that they cannot perceive the virtues of a more liberal education for the younger members of their profession.

My university training was interrupted by war. However, I spent a total of four years at a university studying engineering, but three years' experience since convinces me that this training was neither long enough nor liberal enough.

It is my view that there should be a period of about two years between school and university during which time the young man learns something of the world around him. My most valuable year at the university was that spent after a period of four years in the Army. I quote this experience because it is the views of a very vocal and narrow-minded section of the profession which are more commonly heard.

R. E. D. BURROW, B.Sc.

SIR,—Professor C. E. Inglis in his presidential address to the Institution of Civil Engineers drew attention to the training of engineers and advocated a widening of the basis of the professional training with the following words: "Engineering is now shaping the destiny of civilisation, it has vast potentialities for good or evil, and, side by side with his scientific training, a student should have his interest stimulated towards the humanitarian, the economic, and even the ethical responsibilities of the profession he is about to enter."

In discussing the university education of engineers it is essential to distinguish between "formation" and "instruction". The valuable habit of acquiring knowledge is the principal aid of our nature in reaching for perfection. This knowledge may terminate in a mechanical process, and in something useful; but it may also fall back upon that reason which informs it. In the former case it is useful knowledge and in the latter it is liberal. There are then two principles of education: one dealing with useful arts, the other rising towards general ideas. Those who use knowledge in one way are not likely to use it in the other.

A university is a place of formation rather than instruction—formation implying an action upon our mental nature, while we are instructed in arts and trades, for these are contained in rules committed to memory. Thus a university should employ itself in the education of the intellect to reason well in all matters. Professor Inglis states: "My conception of the primary duty of a university is that it should cater for the needs of those who, without any suggestion of class distinction, are expected to become future officers in the army of civilian engineers—men who are destined ultimately to hold positions of high and varied responsibility. To fulfil this duty, education in the broadest and most liberal interpretation of the term is required. On the other hand, the main preoccupation of technical schools should be to give the specialised training so essential for those who in that army will occupy no less important and far more numerous positions of non-commissioned officers. Education rather than specialised training should be the university ideal, and in this connection it cannot be too strongly emphasised that education is something much wider and more profound than instruction."

Your suggestion that an arts course should precede any science course is undoubtedly the solution. It is interesting to note that the liberal versus the pro-

fessional education was debated at the beginning of the last century by the University of Oxford and the Edinburgh Reviewers, when Dr. Copleston, Provost of Oriel College, said: "It is an undisputed maxim in political economy that the separation of professions and the division of labour tend to the perfection of every art, to the wealth of nations, to the general comfort and well-being of the community. There is no saying to what extent it may be carried; and, the more the powers of each individual are concentrated in one employment, the greater the skill and quickness will he naturally display in performing it. But, while he thus contributes more effectually to the accumulation of national wealth, he becomes more and more degraded as a rational being. In proportion, his sphere of action is narrowed, his mental powers and habits become concentrated, and he resembles a subordinate part of some powerful machinery, useful in its place, but insignificant and worthless out of it. Society itself requires some other contribution from each individual, besides the particular duties of his profession. And, if no such liberal intercourse be established, it is the common failing of human nature to be engrossed with petty views and interests, to underrate the importance of all in which we are not concerned, and to carry our partial notions into cases where they are inapplicable, to act, in short, as so many unconnected units, displacing and repelling one another."

I believe you have every justification for your statement that "a really great engineer or scientist must have a liberal education which puts at his disposal all natural knowledge".

J. L. Bannister.

Cardiff.

Wood as Reinforcement.

Some concrete beams reinforced with strips of wood were recently tested in Hungary and are described in "Engineering News-Record". The concrete had a compressive strength of 5000 lb. per square inch and the wood (silver fir) had an assumed tensile strength of 2000 lb. per square inch. The beams failed due to rupture of the wood, the compressive stress in the concrete being about 4000 lb. per square inch when the wood failed.

The strips of wood were prevented from slipping by steel stirrups and, to prevent swelling when in the concrete, the wood was soaked before being placed. The bond resistance of the wet wood was 100 lb. per square inch, but there was hardly any bond when it was dry. It is recommended that the cover of concrete over the wooden strips should be at least 1½ times the thickness of the strip for interior members. Wood as reinforcement is not recommended for members exposed to severe weather, although precast concrete piles up to 25 ft. long and reinforced with wood (which has about the same elastic modulus as concrete) are used to support a quay wall at Budapest. The piles were highly resistant to damage during driving. Another example of the use of wood as reinforcement is a 30 ft. by 13 ft. concrete caisson.

Prestressed Concrete Convention in France.

The Association Scientifique de la Précontrainte (28, Boulevard Raspail, Paris 7e) announces that a convention on prestressed concrete will be held on October 16, 17, and 18. The convention will comprise lectures and discussions, and visits to a railway sleeper factory at Bonneuil (Seine-et-Oise); the site of the bridge at Villeneuve-Saint-Georges (Seine-et-Oise); quay construction and industrial buildings at Le Havre; underground car park, elevated road, and industrial buildings at Rouen; and the bridge at Moret-sur-Loing (Seine-et-Marne). Details of the arrangements for those who wish to attend the convention may be obtained from the Association.

The Clark Dam, Tasmania

THE new Clark Dam (*Fig.* 1) at Butler's Gorge, Tasmania, is an arch-gravity structure 200 ft. high and about 1,000 ft. long. The thicknesses at the crown are shown in *Fig.* 2. The accompanying illustrations (from the "Constructional Review," March, 1950), show the work during construction. The following notes relating to the design and construction are mainly from papers in the Journal of the Institution of Engineers of Australia from which *Figs.* 2, 5, and 7 are reproduced.

Fig. 1.

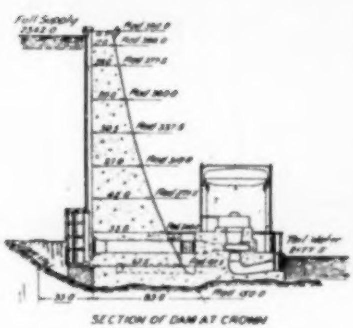


Fig. 2.

The rock at the site is dolerite, and forty 1-in. bores, the deepest of which penetrated 200 ft. below the river bed, indicated that there is a sufficient thickness of rock below the site. There is no definite stratification of the dolerite, but two inclined seams of decomposed dolerite cross the foundations. Brecciation of the material in the thicker seam indicates a fault along the line of the seam. Much of this material has been removed and replaced by concrete. Shallow, or "blanket," low-pressure grouting over the area covered by the dam, and deep, or "curtain," grouting have been done to solidify the rock.

Type of Dam.

Many types of dams were investigated before the final design was settled. Although plenty of rock was available, a rock-filled dam was rejected because of the absence of fine material suitable for the impermeable core. A reinforced concrete cut-off wall in conjunction with slabs on the upstream face of a rock-filled dam were rejected because of the difficulty of ensuring watertightness and stability at the same time as allowing for settlement. A roundhead-buttress dam, although economical, was not adopted because of the complex shuttering and the uncertainties in the design of a structure so much higher than other

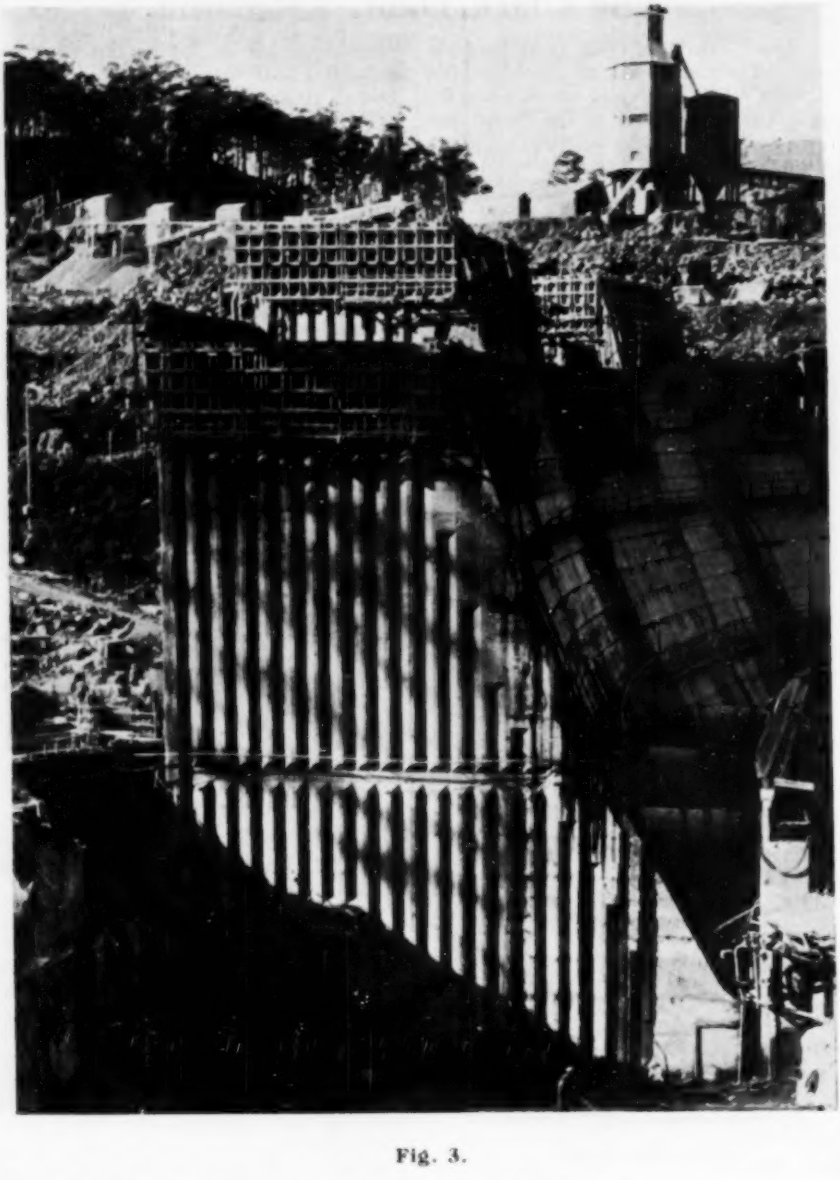


Fig. 3.

dams of this type. Comparison of the estimated costs of a gravity dam, an arch dam, and an arch-gravity dam favoured the last-named type.

A trial-load method of analysis was used in which the pressures are distributed between assumed vertical cantilevers and horizontal arches in such a way that the deflections maintain geometrical continuity. Complete calculations of such a dam require consideration of radial and longitudinal deflections,

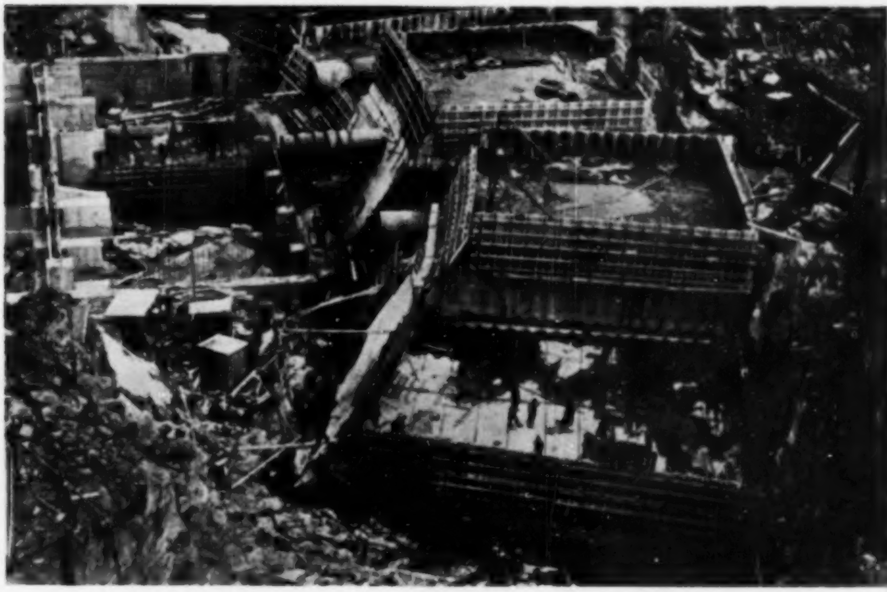


Fig. 4.

the deformation of the rock at the abutments of the arch and the foundation of the cantilevers, rotation about vertical and tangential axes of the centre-lines of the assumed arches and cantilevers, possible cracking of the concrete if the tensile stress exceeds 50 lb. per square inch, and the effect of the heat generated internally. Extreme accuracy is necessary, as the computed deflections are small differences of large numbers. Tests showed that the elastic moduli in compression of the dolerite and concrete made with dolerite aggregate are about $15\frac{1}{2} \times 10^6$ lb. and 5×10^6 lb. per square inch respectively, and these values were used in the calculations.

Since the calculated stresses in the first trials were reasonable, adjustments for radial variations only were made. The dimensions of an existing dam of the same type were modified and used in the first analysis, and small alterations were made for the second analysis upon which dimensions to which excavation could proceed were established. The final analysis, which was made when the excavation was complete, indicated that the probable values of the greatest stresses (per square inch) were 733 lb. (compressive) in the cantilevers and 335 lb. (compressive) and 126 lb. (tensile) in the arch. The corresponding stresses in the second analysis were 670 lb., 300 lb., and 165 lb. per square inch respectively. The greatest deflection in both analyses was 1·2 in. The greatest shearing stress was 217 lb. per square inch.

Vertical radial contraction joints only are provided and are generally at 50-ft. centres. To ensure monolithic horizontal arch action, the joints were grouted when the temperature of the body of the dam was slightly below the mean annual temperature. Details of the joints are given in *Fig.* 5, which also shows a cross section of the dam.

Concrete.

The heat of hydration was kept low by using a modified Portland cement in which the heat generated at seven days did not exceed 70 calories per gramme and at 28 days 80 calories. Because of the risk that the dolerite aggregate might react with the cement, the alkali content of the cement was limited to 0·6 per cent. The cement, of which about 40,000 tons

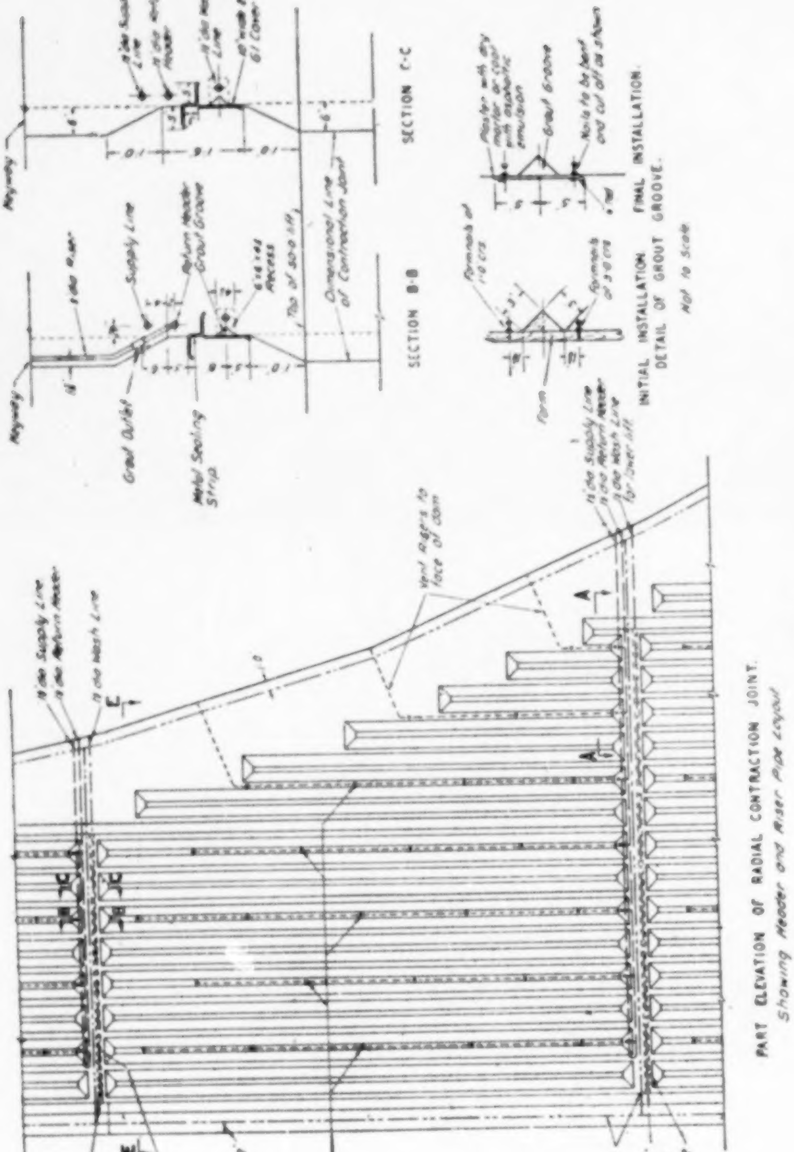

SECTION B-B
SECTION C-C
INITIAL INSTALLATION. FINAL INSTALLATION.
DETAIL OF GROUT GROOVE.
Not to Scale
PART ELEVATION OF RADIAL CONTRACTION JOINT.
Showing Header and Riser Pipe Layout
Grout Outlets
Metal Sealing Strips
Grout Groove
Vent Risers to face of dam

Fig. 5.

were used, was delivered to the site in bags which were split and rumbled mechanically and the contents deposited in a 750-tons steel silo. It is thought that the loss of cement due to this method did not exceed ½ per cent. compared with up to 2 per cent. in the ordinary method of emptying bags. Bulk transport was not used because of the cost of returning the empty containers a distance of 240 miles, partly by road and partly by rail.

The cement content of the concrete is about 10 per cent. of the mixture by weight; it was greater at the commencement of the work and was reduced to 10 per cent. as the control became more efficient. The dolerite aggregate was obtained from a quarry about ¾ mile from the dam, and was crushed to provide three sizes of coarse material and two sizes of fine. The primary properties required of the concrete were durability and impermeability, and a no-slump concrete having these properties would have sufficient strength. The strength of test cylinders and 6-in. cores cut from the dam was about 6000 lb. per square inch at three months.

The materials were proportioned in an automatic weigh-batching plant and the concrete mixed in three 2-cu. yd. tilting-drum mixers producing 2 cu. yd. of concrete per minute. Transport to the site of placing was done in 4-cu. yd. buckets suspended from one of two 10-tons cableways. The concrete was placed generally in 5-ft. lifts, there being usually an interval of eleven days between successive lifts, and was compacted by large pneumatic vibrators. Where concrete was deposited on rock or on well-matured concrete, two or three lifts 2 ft. 6 in. thick were deposited before proceeding with 5-ft. lifts.

It was estimated that the rise of temperature within the concrete would be 46 deg. F. if 5-ft. lifts were placed at intervals of three days, and 31 deg. F. if 2-ft. 6-in. lifts were used. Artificial cooling was therefore installed by embedding ¾-in. gas-pipe in the concrete at about 5 ft. 9 in. centres horizontally in each 5-ft. lift. There are about 600 ft. of pipe in each block between contraction joints. Pipes at 2 ft. 6 in. centres were provided near the faces of the rock. Appliances were provided in the pipes to control the cooling operation and to ensure the turbulent flow necessary for efficient removal of heat. The cooling installation, in which 1½ million gallons of water circulated daily, was designed to reduce the temperature of the dam from

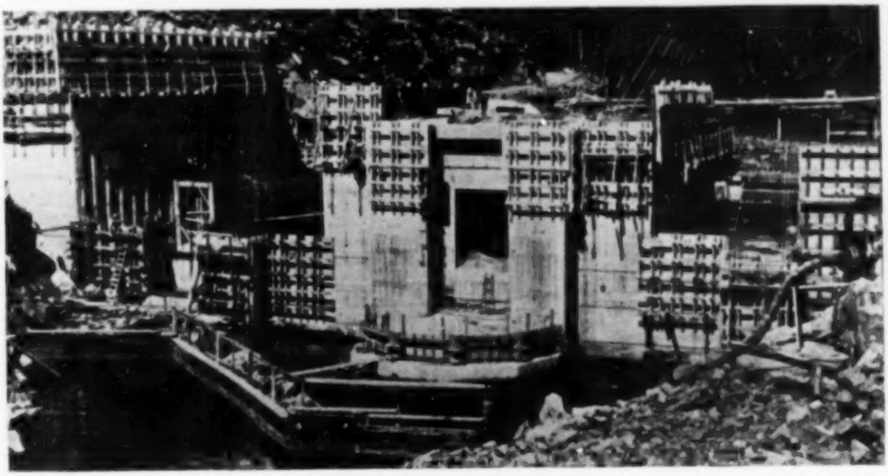


Fig. 6.

89 deg. F. to 45 deg. F. in ten weeks, compared with about four years by natural cooling.

Construction.

Figs. 3, 4, 6 and 7 show the dam in course of construction. *Fig.* 3 shows the piles of aggregate, the mixing plant, and the cement silo. The shuttering and the keyed construction joints are also shown. Other views of the shuttering are shown in *Figs.* 4, 6 and 7; on the left of *Fig.* 4 is seen the foundation of the power-house, and on the right of *Fig.* 4, and in the foreground of *Fig.* 6, the upstream cofferdam.

The cofferdams, which were built of rock and had a clay core, diverted the river to a channel on one side of the site. The diversion could carry a flow of 5000 cusecs., and occasional floods resulting in flows in excess of this amount were allowed to pass through the workings after plant likely to be spoilt by water had been removed. In later stages of the construction, the river flow was passed through the discharge openings in the dam. Concreting of the lower part of the dam proceeded at the same time as excavation, the concrete being kept back so that it was at no time within 50 ft. of rock being blasted.

The masts of each of the cableways were 1200 ft. apart and the head-masts and tail-masts could travel 400 ft. laterally. The two masts of each cableway were kept directly opposite each other or within 40 ft. of this position. The masts were about 68 ft. high; three were constructed of timber and one of steel.

The bags of cement were unloaded by hand from lorries on to an inclined 24-in. belt-conveyor, which could convey 50 tons per hour, and from which the bags were dropped down a vertical steel chute 5 ft. deep. At the bottom of the chute the bags passed along a curved chute to be discharged horizontally into the drum of the rumbler. During passage through the curved chute one side of each bag was slit longitudinally by two knives. The rumbler, which was 10 ft. long, slowly turned the bags over and the cement fell into a hopper below, the empty bags being discharged through the end of the rumbler. Cement-laden air was passed through filters and the salvaged cement passed into the silos. The main cement silo discharged through a vane-feeder into a horizontal screw-conveyor, which discharged into the boot of an elevator that conveyed 15 tons per hour to the concrete mixing installation. Compressed-air was fed into the cone of the silos to facilitate the flow of the cement.

The batching and mixing plant, on which three men were employed, could

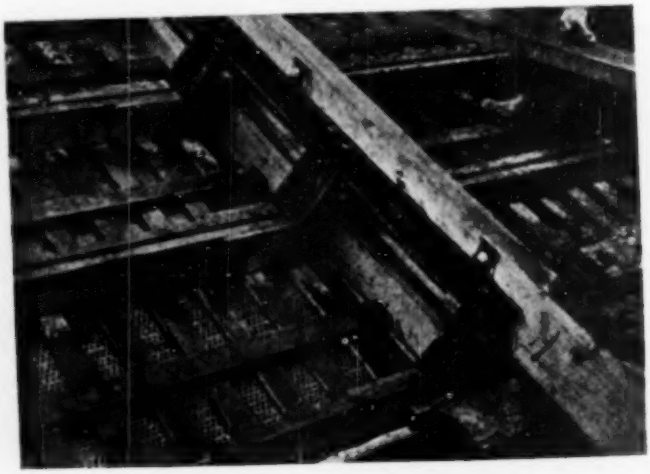


Fig 7.—Porous Shuttering.

produce 120 cu. yd. of concrete an hour, the equipment being designed to handle material to produce batches of 2 cu. yd. of concrete. The greatest quantity of concrete placed at one time was 820 cu. yd., but the average was about 400 cu. yd. Batching was by weight. Three 2-cu. yd. tilting-drum mixers discharged into a central collecting cone, which had no gate as the concrete was retained in the mixers until required. The concrete was discharged into buckets on bogies on a narrow-gauge track passing directly under the cone, and transported thereon to the cableways. The four buckets used for plain concrete in the dam were of 4-cu. yd. capacity, the gate consisting of a piece of rubber belt held in place by rollers, which was closed until a lever was operated to cause the rollers to move and allow the rubber to clear the opening. Buckets of 2-cu. yd. capacity were used for reinforced concrete work, and had a radial-gate having a 12-in. square opening allowing discharge into restricted spaces. Where concrete was to be placed in areas not covered by the cableways, 2-cu. yd. buckets of concrete were transported by motor lorry, the buckets being handled by cranes.

The shuttering in general was timber. The cost, ease of re-use, and surface finish, were very satisfactory. Shuttering for the main part of the dam was of the cantilever type, supported by bolts cast in the two previous lifts of concrete and without internal ties except at the commencement of each part of the work. One disadvantage of the cantilevered shuttering was the relatively large deflection, due to the pressure of the concrete, causing a stepped face. Because it was usual to complete concrete placing against any one shutter as rapidly as possible, the pressure of 5 ft. of wet concrete had therefore to be allowed for in the design of the shutters. Where concrete surfaces would be exposed the shutters, which were generally 12 ft. high and 15 ft. long, were lined with $\frac{3}{8}$-in. waterproof plywood, and were re-used up to 35 times; twelve re-uses were normal for the lining. For curved surfaces, or where little repetition was required, $\frac{3}{16}$-in. plywood was used.

Porous shuttering (*Fig.* 7) was used on the spillway chute, where an accurate smooth finish was important, and comprised 1-in. slats, nailed to main studs. Steel $\frac{3}{8}$-in. screen-plate, fine wire mesh, and butter-muslin were fastened to the slats. Concrete placed on a slope of 1 in 2 tended to "bleed," and water and air coming to the surface passed through the lining of the shutters without carrying away the fine material, and left an excellent surface. These shutters were stripped as soon as the concrete had "bled," that is in three to four hours.

A Large-Span Prestressed Concrete Bridge in Brazil.

Galeao bridge at Rio de Janeiro is a prestressed concrete structure of fifteen spans. Ten of the spans are each 63 ft. 6 in. long, two are 92 ft. 6 in., two are 122 ft. 6 in., and the middle span is 142 ft. 6 in. The reason for the variable spans is that it was originally intended to construct the bridge in reinforced concrete. The bridge carries a road and has a total width of 65 ft. 6 in. The bridge was built in two parts, the first of which was opened in 1949.

Each span comprises nineteen I-beams at 3-ft. 8-in. centres. The top flanges of the beams are 2 ft. 9 in. wide and the bottom flanges 1 ft. 3 in. wide. The web varies in thickness from 4 in. at mid-span to 1 ft. 3 in. at the supports. The shortest beams are 3 ft. 1 in. deep throughout their length, but the beams over the 142-ft. 6-in. span are 5 ft. 7 in. deep at the ends and 6 ft. 3 in. at the middle. The beams are prestressed by the Freyssinet system by cables containing twelve 0·2-in. diameter wires, the number of cables varying according to the length of beam. Each beam in the middle span contains twenty cables, all of which are in the bottom of the beam at mid-span, but some curve upwards at the supports.

Book Reviews.

"Concrete Kerbs: Causes and Prevention of Failures." By J. A. Loe. (London: His Majesty's Stationery Office. 1950. Price 9d. In U.S.A., 25 cents.)

THIS booklet of about twenty pages (Road Research Technical Paper No. 18 of the Road Research Laboratory of the Department of Scientific and Industrial Research) describes investigations made on failures of precast concrete kerbs. The conclusion is that neither the aggregate nor site conditions are common causes of failure, but that the main cause is that the water-cement ratio of the concrete is too high. It is recommended that the average water-cement ratio should be 0·5 to 0·55, and that it should not exceed 0·60.

"Pratique du Calcul du Béton Armé." By G. Magnel. Parts I and II. 5th Edition. (Ghent: Editions Fecheyr. 1949. Prices: Part I, 540 Belgian francs. Part II, 440 Belgian francs.)

THE new edition of the first two parts of Professor Magnel's four-volume work on reinforced concrete differs little from the previous edition as the contents deal mainly with basic principles and matters of a like nature that do not alter or are developed only slowly. In addition to theory, Part I contains the Belgian regulations for reinforced concrete structures, and charts of instructions for concreting in cold weather and for consolidation by vibration. Part II, which deals with continuous beams, the interaction of columns and beams, and slabs supported on four sides, contains some useful matter not generally found in text books, such as consideration in detail of the stresses in the corners of frames, the stresses in columns bending about an inclined axis, and the effect on slabs of flexible supports. Some new tables for slabs spanning in two directions are given. Part II is accompanied by an album of folding charts.

"Durchlaufträger." By A. Kleinlogel and A. Haselbach. 7th ed. Vol. I. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 20 DM.)

THIS edition is to be published in two volumes. The first describes the ordinary theory and the derivation of coefficients for practical application. Four cases of single-span restrained beams and thirty-three cases of beams continuous over two to seven unequal spans are given, with coefficients for the maximum bending moments and reactions due to loads that commonly occur, and with general formulæ for cases of uncommon loads. Numerical examples are given. Volume II, which has not yet been published, will apparently contain the same data but for special cases, such as equal spans.

"Der Haken im Stahlbetonbau." By R. Bauer. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 2.60 D.M.)

IN this booklet of twenty pages tests of various types of hooks on reinforcement bars are described. One of the conclusions is that the anchorage value of standard hooks varies considerably. The decisive factor was found to be the least cover of concrete at the side of the hook. German regulations require a minimum cover of 0·4 in. to 0·8 in., which has proved to be insufficient in many cases. The cover recommended is from 0·8 in. to 4 in., depending upon the strength of the concrete, the quality of the steel, the size of the hooks, and the size of the bar. An anchor-plate was found to be the most economical type of anchorage in structures where a few large reinforcement bars are used.

Packen, Verladen, und Transport von Zement. By E. Plassmann, E. Rubland, and O. Farber. 1949. (Weisbaden: Bauverlag G.M.B. H. Price DM.2.).

THIS is a reprint of an article that appeared in "Zement, Kalk, Gips" in the year 1948, with much additional matter. It describes and illustrates a number of methods of handling cement in bags and transporting cement in bulk by road and rail. The latter part of the work (which is in the German language), and the methods of loading and unloading the containers, are of most interest. The methods described are taken from the practice of the Dyckerhoff Portland-Zementwerke A.G., which is one of the largest cement-making concerns in Germany.

Book Received.

"A Measuring Diagram for Daylight Illumination." By Percy J. Waldram. 1950. (London: B. T. Batsford, Ltd. Price 5s.)

The Design of Multiple-Story Earthquake-Resistant Buildings.

By Dr. H. LEITNER (Quito).

THE reinforced concrete frame of a nine-story building for newspaper offices at Quito, Ecuador, is designed to be resistant to earthquakes. Parts of the third and fourth stories are occupied by a radio auditorium 41 ft. wide, above which are five stories [*Fig.* 1(*a*)] of the same width. The method of designing this part of the structure is similar to that described in the following.

The six superimposed frames are designed to resist the bending moments

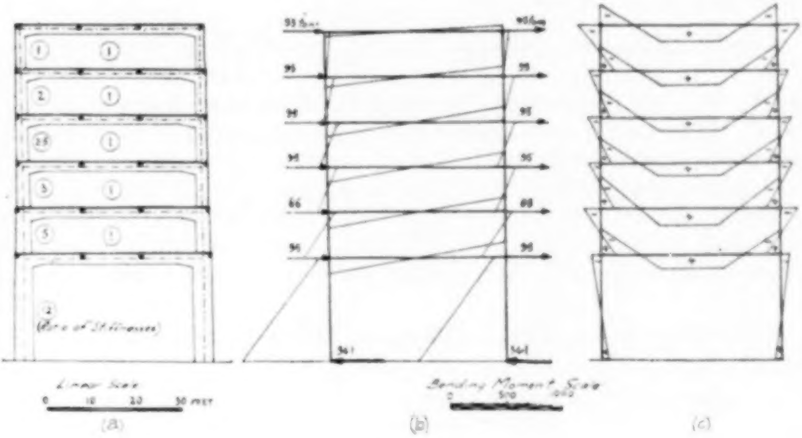


Fig. 1.

[*Fig.* 1(*c*)] and shearing forces due to vertical loads, and those due to horizontal forces [*Fig.* 1(*b*)] induced by earth tremors, and are analysed by a theory derived by the writer and given in " Ingenieria " (Mexico) for October 1944 and January 1945. The acceleration of the seismic wave is assumed to be 3·28 ft. per second per second; therefore each horizontal force, the value of which is given in *Fig.* 1(*b*), is assumed to be one-tenth of the weight of the part of the structure concerned. The figures in circles in *Fig.* 1(*a*) give the ratios *k* of the stiffnesses of the columns, considering that of the beam to be unity. From the bending-moment diagram in *Fig.* 1(*b*) it is seen that the points of contraflexure occur near the mid-height of the columns only in the intermediate stories. In the lower stories the point of contraflexure is towards the top of the column, and in the lowest of the six stories considered there is no point of contraflexure. In the upper stories the points of contraflexure are near the bottom of the column, and in the top story it is at the level of the floor. The variation in the position of the point of contraflexure is due to the variation in the relative stiffnesses of the columns and beams in each story.

Analysis of a Six-story Frame.

A single-bay frame of six stories is eighteen times statically indeterminate, but this degree of indeterminacy can be reduced by first considering the frame as comprising a number of three-hinged statically-determinate frames subjected to the horizontal forces. A building frame of six stories of equal height, and the forces thereon, is shown in *Fig.* 2(*a*). The forces on either side of the line **M–N** are equal and the point of contraflexure in the beam is assumed to be at the midpoint. The number of indeterminacies is therefore reduced to twelve, and is further reduced to six by the fact that the bending moment at the bottom of each column on the left-hand side is numerically equal to, but of opposite sign to, that at the bottom of the corresponding column on the right-hand side of each story.

For the solution of the remaining indeterminacies the general equation for the nth story from the top is

$$-\frac{L_n}{6}X_{n-1}+\left(\frac{H_n}{k_n}+\frac{L_n+L_{n+1}}{6}\right)X_n-\frac{L_{n+1}}{6}X_{n+1}=-\frac{M_nH_n}{2k_n}+\frac{L_n}{6}(M_{n+1}-M_n),$$

where X_n, etc., are the moments by which the three-hinge frame moments are adjusted. Thus for the rectangular frame of span L and equal story-heights H, the equation for each story is:

$$\left(\frac{H}{k_1}+\frac{L}{3}\right)X_1-\frac{L}{6}X_2=-\frac{M_1H}{2k_1}+\frac{2L}{3}(M_2-M_1) \quad . \quad . \quad . \quad . \quad (1)$$

$$-\frac{L}{6}X_1+\left(\frac{H}{k_2}+\frac{L}{3}\right)X_2-\frac{L}{6}X_3=-\frac{M_2H}{2k_2}+\frac{L}{6}(M_3-M_2) \quad . \quad . \quad (2)$$

$$-\frac{L}{6}X_2+\left(\frac{H}{k_3}+\frac{L}{3}\right)X_3-\frac{L}{6}X_4=-\frac{M_3H}{2k_3}+\frac{L}{6}(M_4-M_3) \quad . \quad . \quad (3)$$

$$-\frac{L}{6}X_3+\left(\frac{H}{k_4}+\frac{L}{3}\right)X_4-\frac{L}{6}X_5=-\frac{M_4H}{2k_4}+\frac{L}{6}(M_5-M_4) \quad . \quad . \quad (4)$$

$$-\frac{L}{6}X_4+\left(\frac{H}{k_5}+\frac{L}{3}\right)X_5-\frac{L}{6}X_6=-\frac{M_5H}{2k_5}+\frac{L}{6}(M_6-M_5) \quad . \quad . \quad (5)$$

$$-\frac{L}{6}X_5+\left(\frac{H}{k_6}+\frac{L}{6}\right)X_6 \qquad =-\frac{M_6H}{2\frac{k_6}{2}}+\frac{L}{6}(-M_6) \quad . \quad . \quad (6)$$

The values of M are calculated from $M_n=\Sigma F_nH_n$, and in feet-tons are: $M_1=4{\cdot}4\times 9{\cdot}85=43{\cdot}5$; $M_2=8{\cdot}8\times 9{\cdot}85=87{\cdot}0$; $M_3=13{\cdot}2\times 9{\cdot}85=130{\cdot}5$; $M_4=17{\cdot}6\times 9{\cdot}85=174{\cdot}0$; $M_5=22{\cdot}0\times 9{\cdot}85=217{\cdot}5$; and

$$M_6=26{\cdot}4\times 9{\cdot}85=261{\cdot}0.$$

Substitution of $L=39{\cdot}2$ ft., $H=9{\cdot}85$ ft., and M_1, M_2, etc., in (1) to (6) gives the following:

if $k_1=1$, $k_2=2$, $k_3=3$, $k_4=4$, $k_5=5$, and $k_6=6$.

(1) $22{\cdot}9X_1-6{\cdot}55X_2=70$

(2) $-6{\cdot}55X_1+18X_2-6{\cdot}55X_3=70$

(3) $-6{\cdot}55X_2+16{\cdot}35X_3-6{\cdot}55X_4=70$

(4) $-6{\cdot}55X_3+15{\cdot}5X_4-6{\cdot}55X_5=70$

(5) $-6{\cdot}55X_4+15X_5-6{\cdot}55X_6=70$

(6) $-6{\cdot}55X_5+8{\cdot}18X_6=-1912$

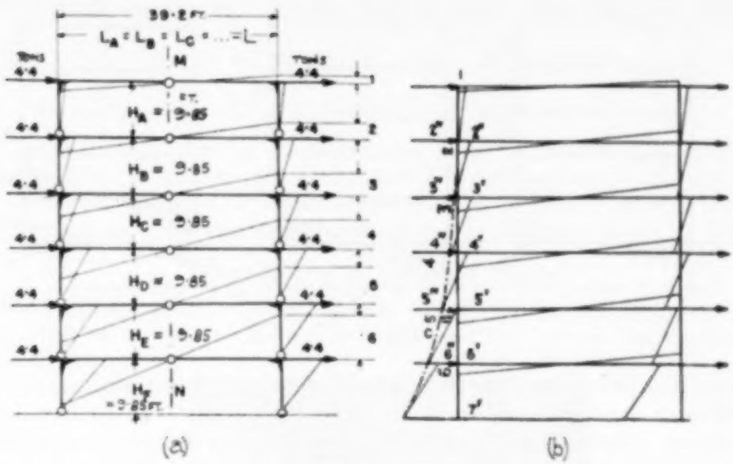


Fig. 2.

The solutions are: $X_1 = -0{\cdot}14$, $X_2 = -11{\cdot}4$, $X_3 = -42{\cdot}3$, $X_4 = -105$, $X_5 = -218$, and $X_6 = -408$ ft.-tons. The net bending moments at the end of each beam and at the head and foot of each column meeting at one joint [*Fig.* 2(*b*)] are obtained from the general expressions for the *n*th joint, namely $M_n + X_n - X_{n-1}$, $M_n + X_n$, and X_n respectively. The numerical values of the net bending moments are:

Bending moment at Point	1	$+ 43{\cdot}5 - 0{\cdot}14 = + 43{\cdot}36$ ft.-tons.
,, ,, ,, ,,	2	$+ 87 - 11{\cdot}4 + 0{\cdot}14 = + 75{\cdot}74$ ft.-tons.
,, ,, ,, ,,	2′	$+ 87 - 11{\cdot}4 = + 75{\cdot}6$ ft.-tons.
,, ,, ,, ,,	2″	$- 0{\cdot}14$ ft.-tons.
,, ,, ,, ,,	3	$+ 130{\cdot}5 - 42{\cdot}3 + 11{\cdot}4 = + 99{\cdot}6$ ft.-tons.
,, ,, ,, ,,	3′	$+ 130{\cdot}5 - 42{\cdot}3 = + 88{\cdot}2$ ft.-tons.
,, ,, ,, ,,	3″	$- 11{\cdot}4$ ft.-tons.
,, ,, ,, ,,	4	$+ 174 - 105 + 42{\cdot}3 = + 111{\cdot}3$ ft.-tons.
,, ,, ,, ,,	4′	$+ 174 - 105 = + 69$ ft.-tons.
,, ,, ,, ,,	4″	$- 42{\cdot}3$ ft.-tons.
,, ,, ,, ,,	5	$+ 217{\cdot}5 - 218 + 105 = + 104{\cdot}5$ ft.-tons.
,, ,, ,, ,,	5′	$+ 217{\cdot}5 - 218 = - 0{\cdot}5$ ft.-tons.
,, ,, ,, ,,	5″	$- 105$ ft.-tons.
,, ,, ,, ,,	6	$+ 261 - 408 + 218 = + 71$ ft.-tons.
,, ,, ,, ,,	6′	$+ 261 - 408 = - 147$ ft.-tons.
,, ,, ,, ,,	6″	$- 218$ ft.-tons.
,, ,, ,, ,,	7	$- 408$ ft.-tons.

Position of Points of Contraflexure.

Attention has already been drawn to the position of the points of contraflexure. It is possible to control these positions by increasing the height of the

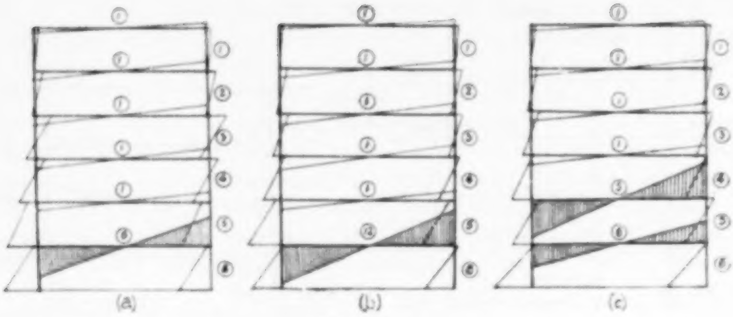


Fig. 3.

lower stories (a procedure which is seldom possible as it entails altering the general dimensions of the building), or by increasing the stiffness of the beams in the lower stories. Both measures cause the point of contraflexure to move towards the bottom of the lower columns and towards the top of the upper columns until in the extreme case the points of contraflexure are at the mid-height of the columns. Another effect is that the bending moment at the foundation of the lowest column is reduced.

The effect of increasing the stiffness of the lowest beam or of the lowest two beams of a six-story frame is shown in *Fig.* 3. At (*a*) is shown the bending-moment diagram if the stiffness of the lowest beam is six times that of the upper beams and the same as that of the column in the bottom story, that is six times the stiffness of the lowest beam in *Fig.* 2. The points of contraflexure are still not within the heights of the bottom and top columns. At (*b*) the stiffness of the lowest beam is made twelve times that of the upper beams, and for this case the point of contraflexure is within the height of the column but still near the top. In (*c*) the lowest two beams are stiffened and a similar condition to (*b*) arises. In all cases the bending moments in the comparatively weak columns are reduced and those in the strengthened beams increased.

To obtain resistance to seismic or other horizontal forces there appear therefore to be two practicable ways of designing multiple-story frames: (1) Beams of ordinary size and columns considerably stronger than is common, as in *Fig.* 1; and (2) with columns of ordinary sizes and with beams of greater strength in the lower stories.

Awards of the Franklin Institute.

The Franklin Institute, Philadelphia, U.S.A., has awarded a Frank P. Brown Medal to M. E. Freyssinet and Professor Gustave Magnel "for their outstanding development of engineering and technique for prestressed concrete members resulting in radical improvement in the design and construction of bridges, buildings, and other structures." The medals will be presented on October 18, at the Institute.

Construction with Moving Forms.—VII.*

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

SPECIAL APPLICATIONS.

IN the preceding articles the structures considered are mainly silos, bunkers, and similar cellular structures. In this article the application of moving forms to the construction of open or frame structures and buildings, to walls containing openings, and to structures of different shapes at different levels, is considered.

Structures of Variable Shape.

In many structures which are mainly suitable for moving-form construction, parts at the top or bottom may not be suitable for this type of construction. Two

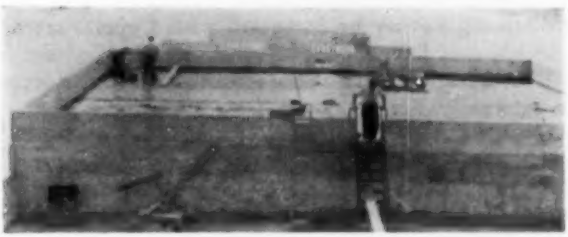


Fig. 39.—Top : Columns built with Fixed Shutters. Right : Bottom Slab built with Fixed Shutters. Moving Forms used for Upper Parts.

examples are shown in *Fig.* 39. The large columns supporting a grain silo and the bottom slab of the bins were constructed with ordinary column and floor shuttering and the moving-form construction commenced at the top of the slab. The parapet above the roof is set in from the face of the main walls, and the concrete was deposited in ordinary fixed shuttering.

It is possible, however, to construct moving forms so that they can be used for columns below the regular cellular part of the structure (the bottom slab of

* Concluded from March to August, 1950.

the bins being constructed later) and for a part of the structure that is set back above the cellular part, as well as for the cellular part. An example of the formwork for a cellular structure that changes in external dimensions near the top is shown in *Fig.* 40. The set-back is allowed for in the original construction of the forms. If the height of the set-back is, say, only 15 ft. or 20 ft., the adaptation of the moving forms may be more expensive than using fixed shutters for the upper part. *Fig.* 40 shows how moving forms can be adapted to suit a set-back by incorporating in the forms for the lower parts the forms for the walls of the set-back. The thick lines denote the walls being concreted and, to restrict the concrete to the correct forms, stop-ends are provided which shut off the parts of the forms not required. When the level at which the set-back occurs is reached, the stop-ends are removed and others put in where required, and concreting

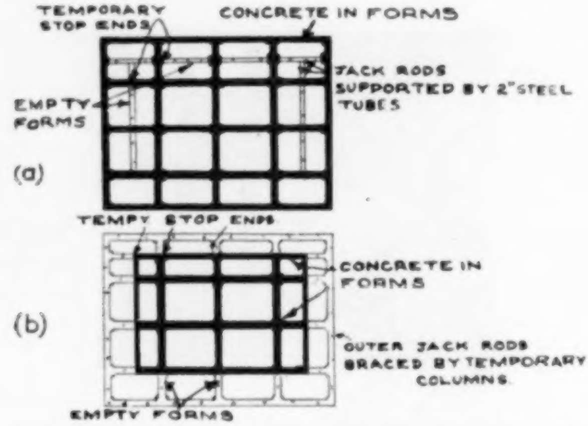


Fig. 40.—Moving Forms for Structure with Upper Part Set Back.

recommences. For the upper part of the structure the outer forms go up empty but are braced internally.

Open Buildings with Floors.

It is not practicable to construct ornate structures in reinforced concrete with moving forms but, with the co-operation of the designer, it is quite feasible to build industrial structures of open construction by suitably modifying the forms. The main requirement is to reduce the features to a plainness and regularity compatible with the requirements of moving-form construction. A wall must be of uniform thickness throughout its height. A column should preferably have the same size in each story, although by inserting packing in the forms reduction in size can be effected. Corresponding beams in each floor must be of the same width, but the depths can be different, if necessary, in each floor. In designing a structure for construction with moving forms the requirements mentioned in an earlier article in connection with the reinforcement must be noted, and the engineer must consider the suggestions of the contractor who is to undertake the work. Experience shows that, for structures not less than 50 ft. in height, open structures are as adaptable for moving forms as are cellular structures.

Open buildings have the disadvantage that the forms are not propped laterally by internal walls but, in a building in which there are internal columns, support can be obtained against the columns. Complete lateral rigidity of the forms must be obtained, otherwise the wall may have considerable variations in thickness and may even be bowed in plan. When the moving of the forms has started, alteration to the forms to overcome a defect is expensive.

Procedure of Concreting.

Multi-story Structures.—As with all moving-form operations, the planning of the work must be considered in the smallest detail. In the case of, say, a warehouse of several stories, the method is first to design the forms for the concreting of the external walls and internal columns and walls and beams. The movement

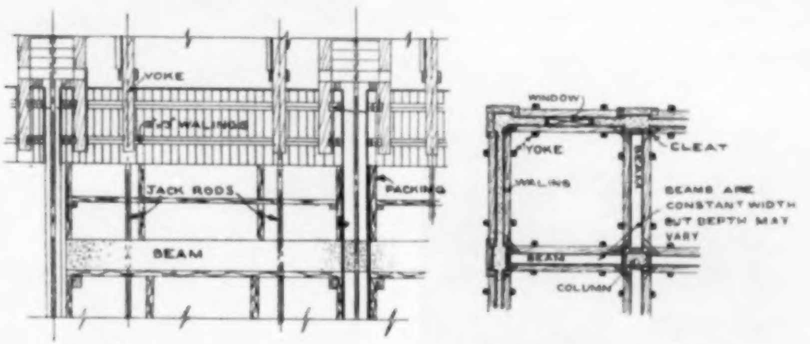


Fig. 41.—Moving Forms above the Level of a Floor of an "Open" Structure.

starts at the ground floor and the walls and columns in the bottom story are constructed. When the forms reach the level of the first floor moving is stopped, the beam soffits are placed in position, and the floor beams are concreted. Then the forms move upward again to the second floor, the first floor is shuttered, and the floor slab is concreted when the moving forms have risen sufficiently to leave working space. The second and subsequent floors are constructed in a similar manner. The procedure depends on the provision of the beam sides in the moving forms so that expeditious concreting of the beams can be achieved. When the floor beams are concreted, the slab can be constructed at any time, but the fact that the beams are already in position ensures a support for the shuttering being readily obtained. The floor slabs should, however, be concreted as soon as possible after the completion of the beams in order to provide lateral support to the walls. Where there are no beams or floors it may be preferable to build the columns with fixed shutters, the walls being braced as described later for open buildings without internal columns.

"Open" Structures.—It has been previously mentioned that, for cellular structures, moving-form construction must not stop, but in the case of open structures the area of the walls is small compared with cellular buildings and consequently the "drag" of the forms on the concrete is proportionally small. The diagram on the left-hand side of *Fig.* 41 is a section through moving forms after the beams of the floor below have been concreted and the forms have risen

above the level of the floor. The steel jack-rods, which between the beams are not encased in concrete, are braced to prevent them buckling. This measure is most necessary since, in the construction of bins, the jack-rods are supported in the wall immediately the concrete in and below the forms has hardened. The strain on the jack-rods is considerable and, even in open structures where the load to be raised is less, the rods tend to buckle if they are not propped. The right-hand side of *Fig.* 41 shows a plan of moving-form shuttering for the construc-

Fig. 42.—Construction of "Open" Building.

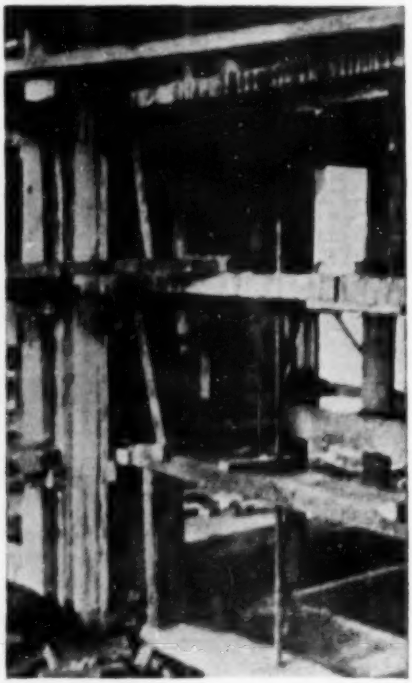


Fig. 43.—Construction of "Open" Building.

tion of beams, columns, and external walls of an open building and *Figs.* 42 and 43 show the construction of such a building in progress. In *Fig.* 43, the method of supporting the jack-rods between floors is shown.

The arrangements for concreting open structures are similar to those for cellular structures, but as the rate of climbing is slower the workmen and supervisors have more time to deal with the difficulties that are bound to occur in all moving-form construction.

Open Structures without Internal Columns.—When an open structure has no internal columns the wall forms can be supported by built-up timber trusses. *Fig.* 44 is an isometric view and a plan of an arrangement suitable for buildings which are not more than, say, 30 ft. wide. The method of supporting

the jack-rods would be as in *Figs.* 41 and 43, or tubular steel scaffolding can be used for this purpose as shown in the plans and elevation in *Fig.* 45.

Where openings in walls occur for windows, doors, and similar purposes, the forms are blocked out by placing timber boxes or frames (*Fig.* 41) at the correct

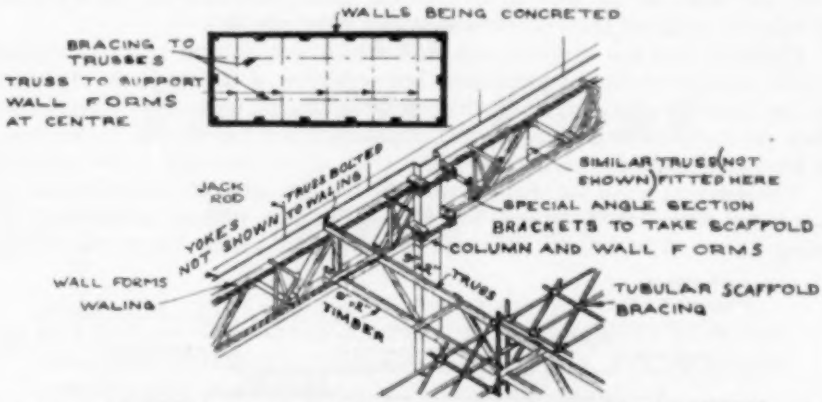


Fig. 44.—Supporting Forms for Open Structure without Internal Columns.

Fig. 45.—Use of Tubular Steel Scaffolding.

position in the forms. If jack-rods pass through the opening, it is necessary to construct temporary concrete columns, if possible at not more than 10-ft. to 15-ft. intervals. *Fig.* 46 shows this type of construction which makes the external jack-rods stable.

TANKS AND CHIMNEYS.—The rise of moving forms for cylindrical reinforced

concrete tanks is similar to that for circular cellular structures, but the standard of organisation need not be so high because a tank is generally a smaller structure. Many tanks have been built with moving forms, some of which are rather crude forms of vertically-sliding shutters. Unless the height of the tank is at least 30 ft. and there are at least three identical tanks, moving-form construction is not more economical than construction with fixed shutters.

Chimneys are tall vertical structures which would seem to be particularly suitable for moving-form construction, but reduction of the thickness of the wall and the diameter of the shaft in the height of the chimney cause complications which generally make moving forms unsuitable. An example of a chimney of this kind constructed with moving forms (*Fig.* 47) is described in the following.

The lower 33 ft. of the shaft around the flue-openings and other breaks were constructed with fixed shutters, but a height of 246 ft. was constructed with moving forms. The internal diameter is 11 ft. 7 in. at the top of the footings

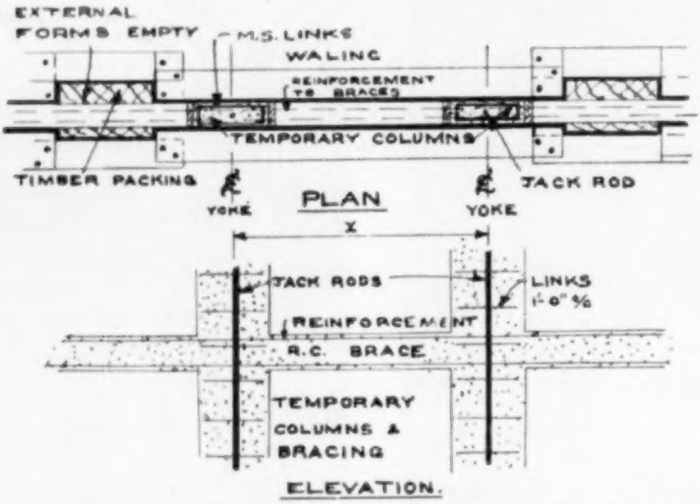


Fig. 46.—Temporary Concrete Columns to Support Jack-rods at Wall Openings.

and 8 ft. 3 in. at the top of the shaft. The thickness of the wall varies from 18 in. at the bottom to 6 in. immediately below the coping. Therefore, in addition to the vertical motion of the forms, two other motions were required to reduce the diameter of the shaft and the thickness of the walls as the forms proceeded upwards. The forms were constructed partly of steel, and comprised six uprights inside the shaft to which was attached the internal sheeting, and which were kept in their correct position by two sets of six threaded horizontal tie-rods. One set was just above the level of the concrete and the other about 8 ft. higher. By tightening the horizontal rods systematically as the forms were jacked up, the internal diameter was reduced as required. Six external uprights were provided and were staggered in relation to the internal uprights, and were connected to them indirectly through a link motion. The lengths of the links were so proportioned that the thickness was reduced automatically as the internal diameter was reduced. The sheeting was attached to the soldiers and was 4 ft. deep, and was in pieces of a constant width. The boards were 1 in. thick. In

moving up the shaft each piece supported a strip of concrete of constant width extending the whole height of the chimney. To support the tapered strips of concrete between the sheets, sheet-iron plates overlapping the wooden sheeting were provided and were kept moving by being hooked loosely over the top of the sheeting. This arrangement allowed the pieces of wooden sheeting to approach one another easily by sliding behind the metal as height was gained.

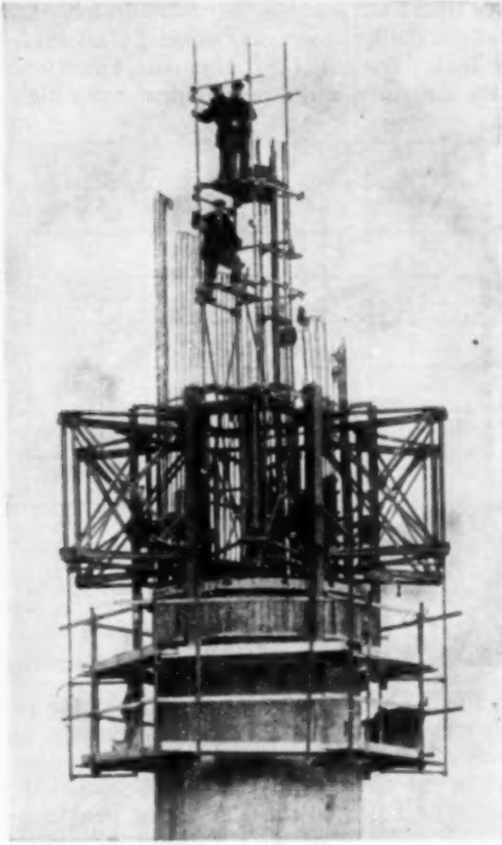


Fig. 47.

Two scaffolds attached to the outside of the plates were able to contract automatically as the diameter of the chimney diminished. The upper scaffold was used by the steel fixers and the lower one, which was reached through a manhole in the upper one, was used by men making good any blemishes in the concrete and fixing attachments for lightning conductors, steeplejacks' ladders, and inspection boxes. The vertical reinforcement bars were carried to the top by a hoist inside the shaft, but the shape of the circular horizontal bars, of which there are three in each ring, prevented them from being handled in this way, and therefore they were hauled up on a snatch-block and rope outside the shaft. The concrete was lifted in small pails by the hoist. An average depth of 3 in. of concrete was deposited in the forms every hour, that is 6 ft. in 24 hours, this

being about the greatest rate of progress possible for such a slender shaft using ordinary Portland cement in winter in England. The work was carried out in three shifts of 8 hours each per day, working seven days per week.

Cost of Moving-Form Construction.

In this and previous articles it is stated that for low structures moving forms are more expensive than fixed shutters, but for walls more than 50 ft. high there is a saving, especially as the deck serves as a working platform and also as shuttering to the soffit of the roof. The initial cost of moving forms is the same irrespective of the height of the structure, and the equipment has a high salvage value.

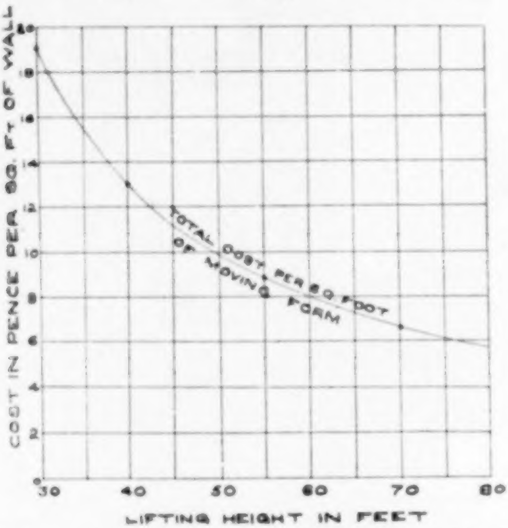
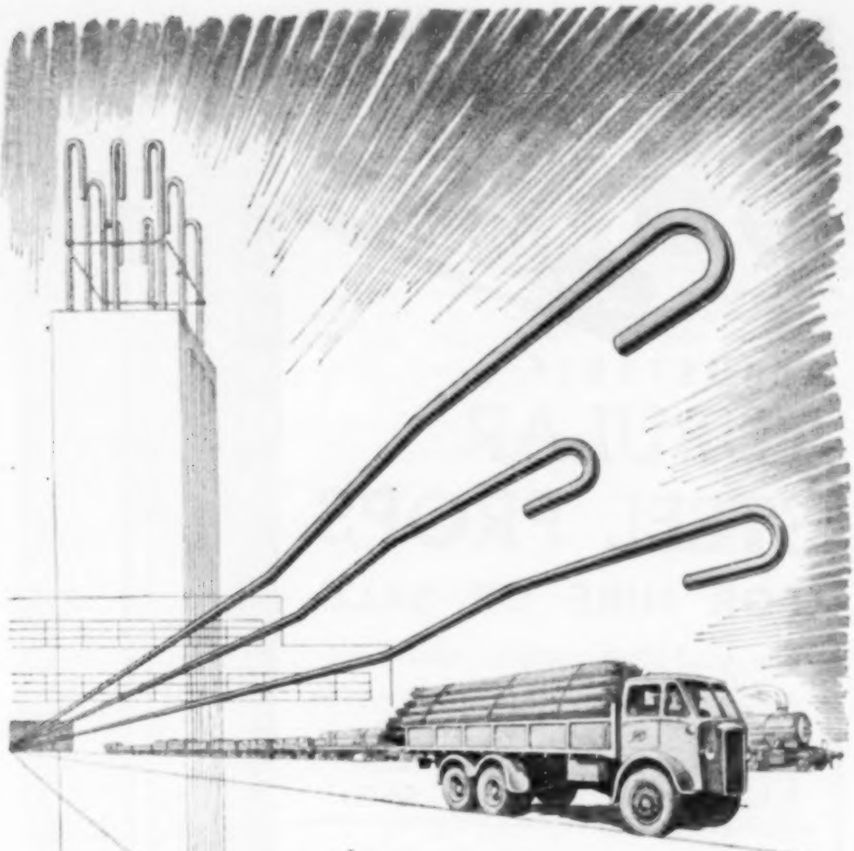


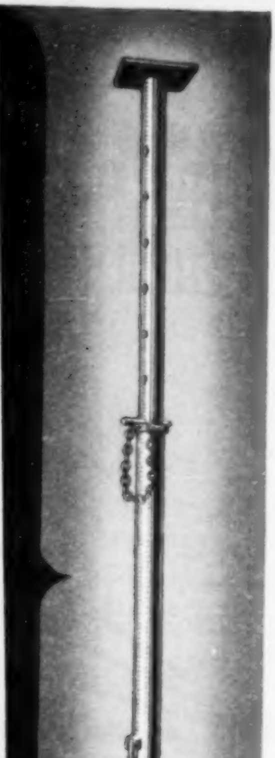
Fig. 48.—Relation between Cost of Forms and Height of Structure.

The curve in *Fig.* 48 shows the relation between the probable cost of the forms and the height of the structure. It is seen that the cost for the forms is reduced very quickly above a height of 30 ft. and becomes more constant at 80 ft. Therefore the cost of the forms is inversely proportional to the height of the structure. The costs are based on those for a coal bunker erected in 1946 and, although not based on several structures, indicate the general trend of costs of moving forms. In this case steel forms instead of timber laggings were used. The same amount of jacking has to be undertaken for unit area of wall whatever the height. It would be reasonable to expect that the variation of cost of fixing the reinforcement would be similar to that of jacking, and the cost of hoisting would increase as height increases.

Hollow-Block Slabs.

A REVISION recently made to British Standard Code of Practice CP. 114,103/1950 (see this journal for April, 1950) requires that the minimum thickness of the topping of a floor slab constructed with permanent blocks be 1 in. (instead of $1\frac{1}{4}$ in.) if the distance between the ribs does not exceed 18 in.





Concrete of High Strength

In the course of a lecture given at the Building Exhibition held last year in London, Dr. A. R. Collins, A.M.Inst.C.E., described some of the principles of making concrete having compressive strengths up to 10,000 lb. per square inch. The following is a part of the lecture.

High-strength concrete requires a large proportion of cement, carefully selected aggregates, and good workmanship. If high strength is required at an early age, heat curing is generally necessary, or high-alumina cement or a mixture with on the outside and progresses inwards so that if hydration is incomplete the cores of the grains remain unaffected but the cementing value of the outer layers is unimpaired. However, the cementing value and strength of the cement paste must be affected by the presence of the small voids formed by the uncombined water. A low water-cement ratio generally requires a rich mixture to give sufficient workability, and to prevent excessive moisture movement and shrinkage very rich mixtures must have a low

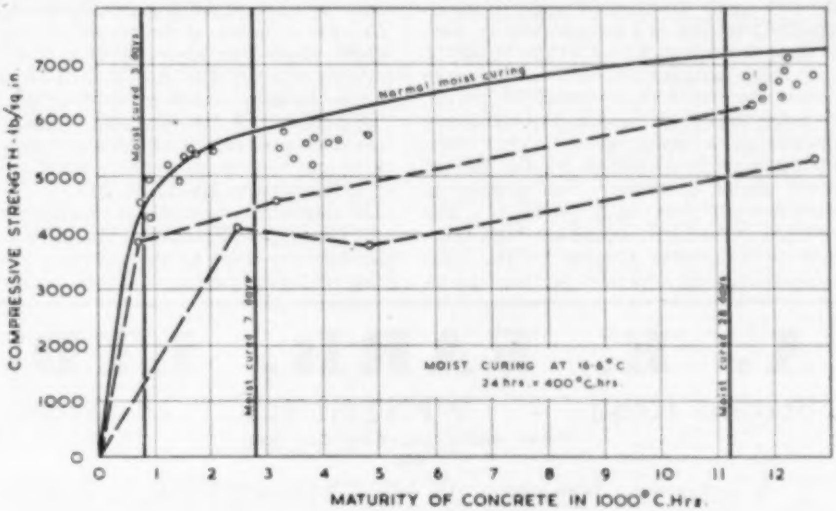


Fig. 1.

a high proportion of Portland cement and a low water-cement ratio should be used. It was explained that knowledge of the production of very high strengths is very fragmentary.

Tests show that when Portland cement hardens it does not combine with more than about half the weight of water in the mixture, the other half of the water forming fine pores in the concrete. For complete hydration the cement must combine with 20 per cent. to 25 per cent. of its weight of water. Therefore if the water-cement ratio is less than about 0·44 the cement does not in the early stages become fully hydrated. This fact does not, however, appear to affect the strength, as hydration of the grains of cement begins water-cement ratio. Although the type of aggregate does not seem to affect the one-day strength, it does seem to affect the strength as the age increases and as the water-cement ratio decreases. In recent tests the highest strengths were obtained with granite coarse aggregate and natural sand whilst the lowest strengths were obtained with the natural uncrushed gravel. The tests also showed that for the same water-cement ratio lean mixtures gave higher strengths than the richer mixtures. The consistency of the mixtures producing concrete of very high strength is generally too dry to be measured by the slump cone and for such mixtures the compacting factor was used in the tests to assess the workability.

The results of the compacting-factor tests also showed the superiority of granite aggregate, other aggregates giving results roughly in the same order as in the strength tests. It would be unwise, however, to draw the conclusion that granite aggregate is always better than flint gravel until more materials have been tested.

Heat Treatment.

The rate of hydration of cement, provided that water is always present, will within limits increase as the temperature is raised. This ability to increase the rate of hardening of concrete by heating is sometimes used to obtain high strengths at early ages, the process being to maintain the concrete at a temperature of, say, 70 deg. to 80 deg. F., or to use steam to raise the temperature of the concrete almost to the boiling point of water. In some processes, which require costly equipment, a temperature higher than boiling point is obtained by the use of steam under pressure; this process is suitable only for small products. The strength of concrete cured at high temperature is greatly affected by the time between mixing and the application of heat, the size of the concrete member, and the rate of heating. As the strength of concrete depends on time and temperature, the total curing value can be represented by the result obtained by multiplying the number of hours of curing by the temperature. Thus, curing for 100 degree-hours may mean two hours at 50 deg. C. or four hours at 25 deg. C. It has been found that within reasonable limits the strength of concrete cured for the same number of degree-hours is constant even if the actual times and temperatures are varied. It follows that high-temperature curing does not produce higher strength but merely decreases the time required to reach a certain strength. This phenomenon is illustrated in *Fig.* 1, which shows the strength of a concrete plotted against the number of degree-hours of curing (called in the diagram the "maturity" of the concrete). The full line is that obtained at normal temperatures and the circles are for steam curing at temperatures up to 90 deg. C.

If concrete is heated too quickly there will be large differences in temperature through the body of the concrete which may result in cracking. If the tempera-

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ture is raised too high too soon after mixing, a slower rate of increase in strength and later a decline in strength occurs, as shown by comparing the lower dash-line in *Fig.* 1 with the upper dash-line.

If the concrete product is large, the heat must be applied more slowly and for a longer period than with small products, and in a particular case tests should be made to determine the most economical heat cycle as it is clearly uneconomical to apply more heat than is necessary.

If the concrete is to be stressed a few hours after mixing, the heat must be applied as soon as possible and the temperature raised as high as possible by placing the product in a steam oven or in hot water, but this process cannot be applied to products having cross-sectional dimensions of more than a few inches. If twelve hours or more are available for curing, the temperature can be raised more slowly by the ordinary processes used in a precast products works, and the greatest temperature reached need not be so high. Heat curing can sometimes be used with advantage for work that is not to be stressed for several days, since in this case a leaner mixture than otherwise required can be used.

Compaction.

To obtain concrete of very high strength compaction is often the main requirement. In order to use low water-cement ratios with mixtures of reasonable proportions it is necessary to be able to compact concrete of very low workability and vibration is nearly always essential, but in making high-strength concrete powerful vibrators are required. It appears that the best results are obtained by vibrating first at a low frequency and then at a higher one. With mixtures of the very lowest workability, vibration alone ceases to be effective and it is necessary to apply pressure to the concrete to cause the particles to cohere, but it is difficult to say, without much more experience than is at present available, which mixtures require both pressure and vibration. Concrete having a workability represented by a compacting factor of less than 0·6 cannot be placed by vibration alone, and mixtures with compacting factors of less than about 0·75 to 0·80 are not likely to be fully compacted by hand ramming.

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"417," 87s. 6d. in 6-ton loads; paper bags 9s. 6d. extra.
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REINFORCEMENT.—Mild steel round bars (per cwt.): [illegible] in. to 2[illegible] in., 28s. 2d. [illegible] in. to [illegible] in., 29s. [illegible] in., 29s. 8d. [illegible] in., 31s.

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Raking, cutting, and waste, 5d. per lin. ft.
Labour on splays, 3d. per lin. ft.
Small fillets to form chamfers, 6d. per lin. ft.

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